Understanding Longshore Sediment Transport on a Mixed Beach

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Abstract

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The aim of this study is to acquire a better understanding of the short to medium timescale (diurnal to monthly) morphodynamic behaviour of mixed, sand and shingle, beaches. In particular, this project intends to provide a new approach to estimate longshore sediment transport (LST) rates on a mixed beach. In order to do that, the mixed beach at Milford-on-Sea, Hampshire UK, was subject of a novel field experiment consisting on an impoundment technique, where beach profiles were measured daily over two months using RTK-GPS. Meanwhile, the nearshore hydrodynamic conditions were measured collecting wave and tidal data. Then, the short to medium term morphological variability of the beach shoreline was investigated related to the hydrodynamic regime.

Understanding the longshore sediment transport processes is important for applying the adequate coastal management policy. At the same time it is imperative to develop methodologies that can be applied widely to other coastal sites with similar characteristics. Among the different methods for measuring LST rates, the impoundment technique was considered as an appropriate approach field wise because its efficiency for trapping sediments has been demonstrated, mainly within the swash zone, which significantly contributes to the longshore sediment transport.

The results from the analysis of the beach profiles based on the Energy Flux method showed that it is a good approximation to relate beach morphology changes to the hydrodynamic conditions. Five different longshore sediment transport coefficients *k* were obtained using the CERC Equation (1984) considering different approaches for beach volume computations taking into account the effect of the tides. Only one sediment transport coefficient was of a similar order of magnitude than other values found in the literature for coarse sediments. Results of this study shows the importance of the seaward limit of the beach profile when computing LST rates to ensure that the range of action of the sediment transport processes is considered.

A one- line shoreline model was validated using the Milford-on-Sea field data collected in order to assess the capability of the model to predict shoreline changes for a mixed beach. The sensitivity analysis on the calculated sediment transport coefficients shows that higher coefficients over-predict shoreline changes as compared to lower coefficients. The comparison between modelled and measured shorelines shows consistent patterns but a significant shift in the offshore direction.

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Conference publications:

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- Calibration of a longshore sediment transport formula on a mixed beach. Marine Institute Symposium 2009, Plymouth.
- Determining littoral transport rate on mixed beaches using an impoundment technique. International Conference in Coastal Engineering, ICCE 2008, Hamburg.
- Impoundment technique to measure LST Rate on a mixed beach. Young Coastal Scientist and Engineers Conference, YCSEC 2008, Oxford.
- Determining littoral transport in a mixed beach using field data. Marine Institute Conference 2007, Plymouth.

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Chapter 1

1 Introduction

1.1 Understanding Longshore Sediment Transport on a mixed beach

Coastal environments are natural systems susceptible to change under the actions of coastal processes and from the consequences of the development of communities around littoral boundaries. Furthermore, the effect of climate change such as the sea level rise, storm surge events or coastal flooding, increases the vulnerability of coasts. In order to deal with these increasing stresses, more extensive and detailed information concerning shoreline changes, beach morphodynamics and coastal processes is required by local authorities in the UK, and elsewhere in the world, to integrate sustainable activities within the environment under the framework of coastal management plans. In the UK Shoreline Management Plans (SMPs), are documents with recommendations for action based on long term assessments (50-100 years) of the likely effects of coastal processes on the littoral environment.

It is known that beaches are one of the most efficient natural mechanisms for protection the coast, and their configuration and reconfiguration is the result of the dynamic energy balance between the action of waves and tides and the sediments moving along the coastline. In 2003, López de San Román-Blanco noted that mixed and shingle beaches being common features along the South coast of the UK, they were less well understood than sandy environments, although being of considerable interest to coastal authorities. Since then, more attention has been focussed on these mixed and coarse sediment environments, (e.g. Horn and Walton, 2007; Pedrozo-Acuña et al., 2007; Curtiss et al., 2009; Ciavola and Castiglione, 2009; Horrillo-Caraballo and Reeve, 2010; Dickson et al., 2011; Roberts et al., 2013). This evidence led to a better understanding of the influence of the mixture of sediments to the beach behaviour, comparedwith pure sand or gravel beaches. In fact, the research of gravel beaches has increased significantly, particularly, looking at cross-shore processes and the relation between the hydrodynamics and shoreline changes (e.g. Pedrozo-Acuña et al., 2006 and 2008; Austin and Masselink, 2006; Pye and Blott, 2009; Ruiz de Alegría-Arzaburu et al., 2010; Jamal et al., 2010; Williams et al., 2012).

Given a supply of sediment sources, one of the main factors in the long term development of a beach is the longshore component of the sediment transport along the coastline due to the interaction of waves and tidal currents (Rogers et al., 2010). The quantification of coastal sediment transport rates, especially the longshore sediment transport, LST, constitutes one of the most useful pieces of information for long term management in coastal engineering. Spatial and temporal changes in LST along a coastline are inextricably linked to beach profile changes over both, the short and long term (Horikawa, 1988). For instance, measurements of beach profiles are used to estimate variability in relation to meteorological forcing and to monitor the changes of beach volume and the shoreline position (Reeve et al., 2004). The latter, “line of demarcation between the water and the exposed beach” (Komar, 1976), is variously defined depending on the data available to set a datum for the seaward extent (Farris and List, 2007).

The behaviour of mixed beaches in comparison to pure sand or gravel beaches is more complicated by changes to the hydrodynamic processes created by the different hydraulic properties of mixed sediments (Kirk, 1980; Mason et al., 1997). Examples of research into mixed beaches in terms of their dynamics and predictive tools for their behaviour are few and far between: Kirk, 1980; Mason et al., 1997; Mason et al., 2001; Bradbury et al., 2003; López de San Román-Blanco, 2003; Pontee et al., 2004; López de

San Román-Blanco et al., 2006; Buscombe et al., 2006; She et al., 2006; Ivamy and Kench, 2006; Horn and Walton, 2007; Ciavola and Castiglione, 2009 Karunarathna et al., 2012. In these, there is much mention of the need for further research, including the calibration of existing methods derived from laboratory experiments or physical models. In general, gravel and mixed beaches are distinguished from sandy beaches because they present a steep gravel face, with typical gradient 1:6 – 1:8. Consequently, there is a narrower surf zone where refraction processes take place (Kirk, 1980). This type of beach is usually characterized as being reflective due to the high permeability of the shingle fraction (Van Wellen et al., 2000). The harsh conditions of these coastal environments confine the deployment of delicate instrumentation for data recording, and as a result, there is a scarcity of measurements for gravel and mixed beaches with which to predict their morphodynamic behaviour over long time scales (Van Wellen et al., 2000; Bradbury et al., 2003).

There are only a few studies reported in the literature that attempt to calibrate longshore sediment transport (LST) formulae for mixed and gravel beaches comparing with the studies in sandy beaches. These are mainly based on purely field measurements of the longshore sediment transport rates and climate weather conditions. One of the most familiar formulae for LST in coastal engineering is the CERC equation (CERC, 1984) which has been developed for sandy beaches. This empirical equation is based on the Energy Flux method which considers that the immersed weight of the alongshore moving sediment is proportional to the alongshore wave power per unit length of beach (Kamphuis et al. 1986), the proportionality coefficient relating the two parameters is known as the sediment transport coefficient, *k*. This method is explained in detail in Chapter 2; however, it is important to note that the value adopted by *k*, and consequently, the corresponding methods for its estimation, is being subject of discussion by different authors (Bodge and Dean, 1987; Bodge and Kraus, 1991; Stutz and Pilkey, 1999; Pilkey and Cooper, 2002; Cooper and Pilkey, 2004) as it is not a constant value that varies depending of the specific site conditions by which it is applied.

Thus, the questions arise about whether the approach adopted by the CERC (1984) formula is realistic, or should being considered more qualitative rather than quantitative, or whether it is taking into consideration the appropriate physical parameters involved in the sediment transport processes. Therefore, the general applicability of this equation as a predictive tool of the LST rates for engineering management schemes is widely questionable. Nevertheless, further research attempting to improve the capability of this method has been undertaken, this led to the development of new numerical formulations for the estimation of LST rates (Bayram et al., 2007; Tomasicchio et al., 2013).

1.2 Beaches in the context of coastal engineering

1.2.1 Inshore and offshore engineering

From an engineering point of view, in general, beaches constitute the best natural buffer to protect coastlines against flooding and coastal erosion (Rogers et al., 2010). Despite the fact that mixed shingle-sand and gravel beaches are present worldwide, they are of major significance for coastal engineers in the UK (Voulgaris et al., 1994). On one hand, such coastal environments are important as they play a natural coastal defence; on the other hand, they are also relevant for the construction industry related to aggregates extraction activities and coastal replenishment (Voulgaris et al., 1999, Van Wellen et al., 2000), since the soft engineering approach is being adopted as a more sustainable strategy than classic hard engineering methods (Rogers et al., 2010). Thus, further investigations should be focused upon providing guidelines for the best practice in coastal management schemes.

Also, it is noted that in the last years the offshore energy developments like offshore windfarms or wave energy farms, as well as the new nuclear power build, have been increased in the English and Welsh coastal waters. The location, design and layout of those developments are some of the key elements for the assessment of potential impacts on the coastal environment. Then, under the regulations of the Planning Act 2008 established by the Planning Inspectorate and the Marine and Coastal Access Act 2009, for the English inshore and offshore waters, by the Marine Management Organization, the developers should provide an Environmental Impact Assessment within the application, to evaluate the potential effects of the infrastructure on the near coastal communities.

Regarding the marine aggregate activities, given the existing control strategy by the government under its National Planning Policy Framework (NPPF), by the requirement of Local Aggregates Assessment submitted by the planning authorities[1]; and given the relatively long distance of the active extraction areas to the coast, in terms of physical processes, they are hardly likely to cause any significant effect to the coastal environment (Newell et al., 2013).

1.2.2 Shoreline Management Plans

Shoreline Management Plans have their origin in 1994, when the Government stimulated Coastal Groups and local coastal authorities in England and Wales to develop recommendations for the management of coastal defences from a strategic and sustainable point of view (EA website, 2011). This policy is a non-statutory document that sets out a series of guidelines for managing the coasts; there are defined four shoreline management policies: Hold the Line (HTL), Management Realignment (MR), No Active Intervention (do nothing) (NAI) and Advance the Line (A).

[1] Source: http://planningguidance.planningportal.gov.uk/

In order to assess the coastline, the shoreline of England and Wales is divided by the SMPs into cells and subcells; the divisions are made considering coastal type and morphodynamic processes (Figure 1.1).

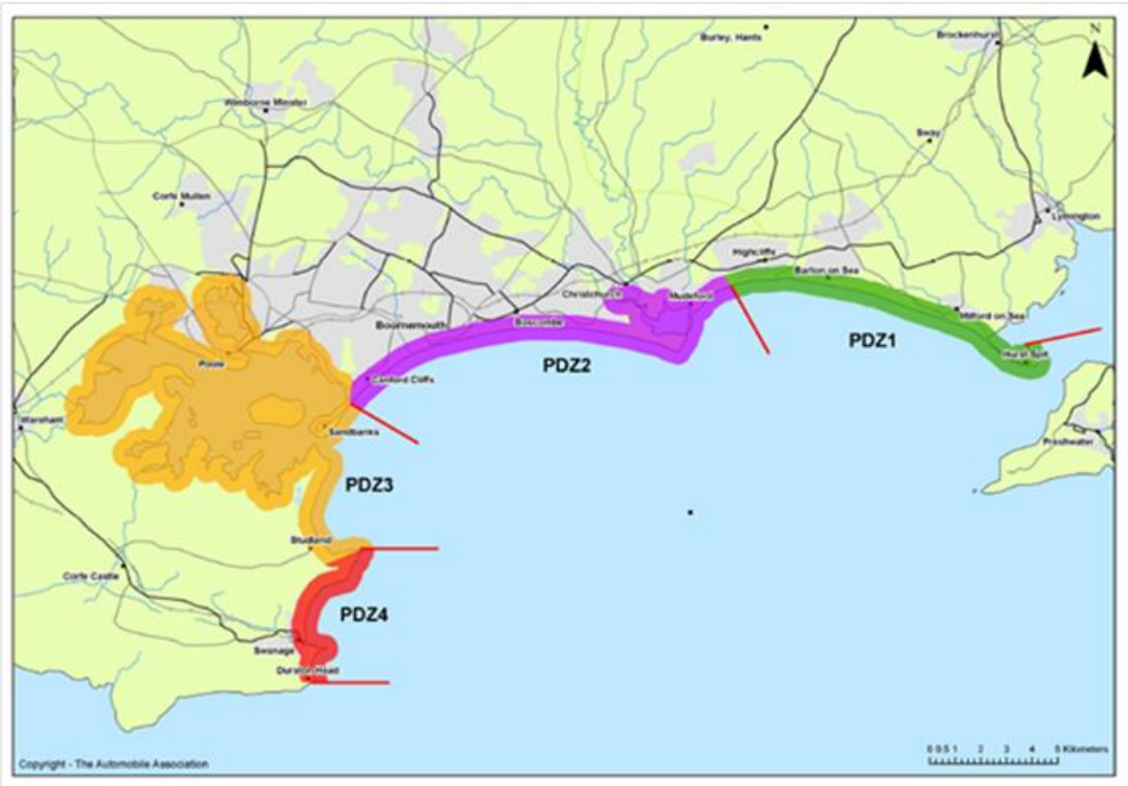


Figure 1.1 Location of Sub-cell 5f corresponding to Poole and Christchurch Bays Shoreline Management Plan 2, South UK. The sub-cell is divided in four PDZs, (image courtesy of Poole and Christchurch Bays Coastal Groups, http://www.twobays.net).

The first generation of SMPs, known as SMP1, identified the baseline and strategy, i.e. the current situation as a reference, providing the route to the decision makers for carrying out the management changes where appropriate looking at the future of 30-50yr; they were completed in 1999. Later on, the second generation of SMPs, SMP2, were produce as a review and update of the SMP1; they differed from the SMP round 1 in that the assessment of the coastal morphodynamics form the basis of sustainable policy development and they looked at +50-100yr. (Rogers et al., 2010). The SMP2 considers the sub-cells also divided into Policy Development Zones (PDZs); each PDZ is divided into Management Areas (MAs) and finally the MAs into Policy Units (PUs).

While the PDZs are considered in the process of developing policy, the final definition of the policies and the coastal management consider the MAs and PUs.

It is acknowledge that coastal steepening, or foreshore narrowing, has been occurring from the early 1900s, however, in the UK it has been understudied (Taylor et al., 2004). In a macroscale analysis of coastal steepening around the coast of England and Wales, Taylor et al. concluded that 61% of the coastline investigated has shown a steepening trend, being the dominant tendency on each of the west, south and east coasts. This process can be assessed by the variability of the tide water levels and the evolution of the beach cross- shore profile; therefore, Dornbusch et al. (2008) after evaluating the coastal narrowing along the Southeast coast of England, concluded that investigations in this subject should examine in detail the data sources and the errors associated with that data, and they disagree with the findings by Taylor et al. (2004), questioning whether coastal narrowing is present at a regional or national scale. Therefore, the interest for coastal steepening processes has been increased recently, and discussed, by coastal managers.

By 1990, Powell noted that inshore wave climate predictions had been improved, which was beneficial for the coastal structure designs but not for the design and management of shingle beaches. Thus, Powell (1990) encouraged by the need for a better methodology from the beach design perspective, developed a model to predict the short term profile response for shingle beaches. The results suggested that this tool was a promising technique to be applied in coastal management programmes and it led a way for further research.

Afterwards, in the late 1990s, mixed beach research was identified as an active topic for growth, since little research was available in this field (López de San Román-Blanco, 2003). López de San Román-Blanco (2003), further to her experimental and field studies in coarse-mixed beaches, suggested areas for future investigation, those included: the highlighting of differences between sandy, mixed and gravel beaches; the significance of the grain size distribution; other characteristics that determine both the modality of sediment transport and morphodynamic behaviour. According to the conclusions of Trim et al. (2002) from the results of a laboratory study to a shingle beach, the role of the tide was identified as an important forcing mechanism on beach profile development, which had been hitherto downplayed. In conclusion, this work identified the need of developing empirical predictive tools for mixed beaches as well as large data sets for validation purposes.

1.3 Introduction to the littoral zone

In order to understand the beach morphodynamics and the relevant processes involved, it is helpful to start describing the littoral zone and beach morphology. The littoral zone is considered the area that extends cross-shore, from the exposed beach to a water depth at which, the surface waves hardly have effect on the bottom, and in consequence, there is minor sediment transport activity (Komar, 1998). Then, the littoral zone comprises the nearshore zone, Figure 1.2; this environment is where the key physical processes such as waves and currents, which affect the beach morphodynamics, take place. This region extends seawards from the shoreline to just the offshore boundary of the breaker zone; the latter, comprises the part where the waves, approaching from offshore, become unstable and break dissipating the energy. As a consequence of the wave breaking, the processes known as wave set-up and set-down take place in the surf zone, and outside seawards of the breaker zone respectively. The next zone landwards is the surf zone; this is the region where waves propagates after breaking and energy dissipation takes place while the wave height decreases progressively. The beach slope is the factor that mainly controls the presence and the width of the surf zone; then, in minor effect, the stage of the tide (Komar, 1998). Also, the beach slope is characteristic of each beach and it is related to the grain size (Kamphuis 2000).

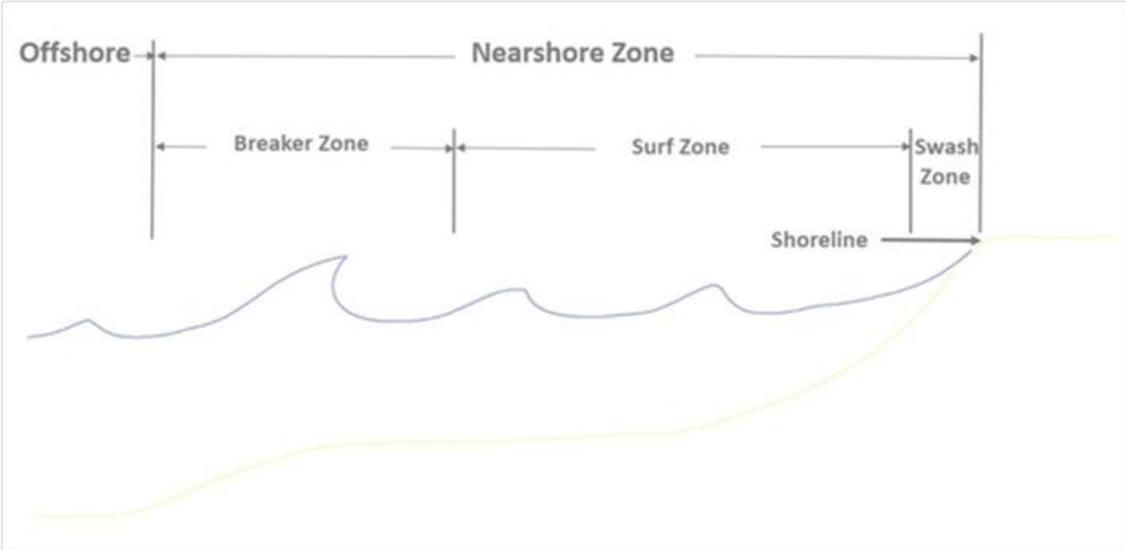


Figure 1.2 Schematic representation of the nearshore zone nomenclature which includes the breaker, surf and swash zones, relevant for the description of coastal processes (after Komar, 1998).

Finally, following the surf zone landwards, there is the swash zone; it is the portion of the nearshore zone where the water moves along the foreshore, explained below, swashing run-up and run-down the beach face. Chapter 2 describes in more detail the physical processes occurring in the nearshore region and their implications in the shoreline changes and sediment transport.

The beach profile is defined by Kamphuis (2000) and Dean and Dalrymple (2004) as the variation of water depth with distance offshore from the shoreline; its shape will be determined by its response to the forces of waves and tides acting on the sediments. This cross-shore section comprises the backshore, foreshore and inshore areas, Figure 1.3. The inshore region of the profile corresponds to the breaker and the surf zones; the foreshore is used to denote the sloping section of the profile that comprises the swash zone; it extends between the upper limit of the swash zone at high tide, or the berm crest if it is formed, and the low water mark of the run-down at low tide (Komar, 1998).

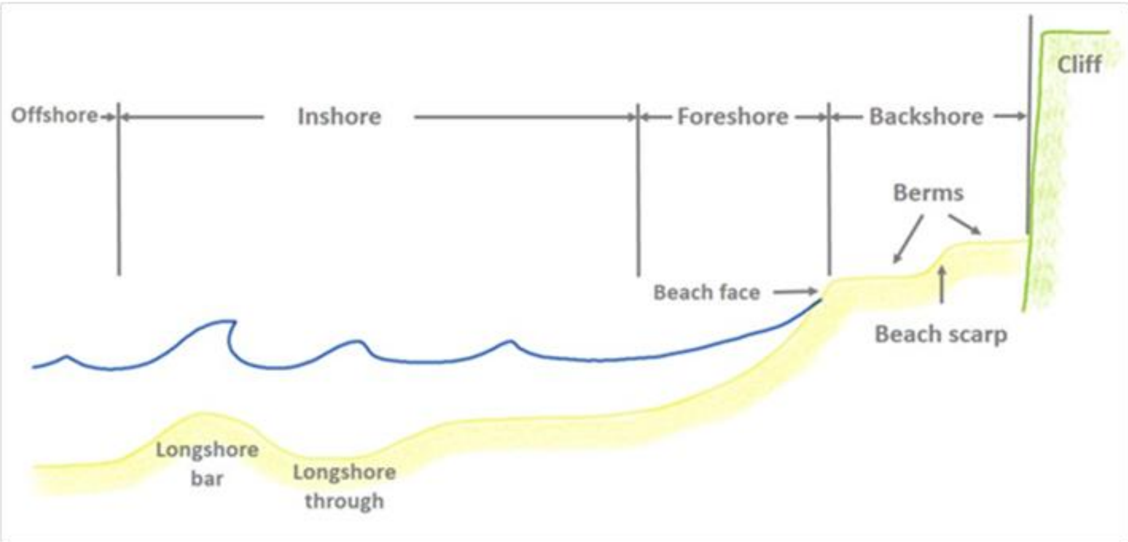


Figure 1.3 Schematic diagram representing the nomenclature used to define the beach profile, (modified from Komar, 1998).

The backshore corresponds to the section of the profile that extends landwards from the limit of the foreshore to the place at which there is a physiographic change as e.g. the toe of a cliff, vegetation or dunes. Masselink (2011) defines the beach face as the planar and relatively steep upper part of the beach profile where swash processes take place. The berm is the nearly planar section that extends landwards from the beach face separated by the berm crest. The berm is an accretionary feature resulting from the deposition of sediments at the landward extreme of the wave run-up (Masselink, 2011). In some cases, as it is affected by wave action, in large tidal ranges it is possible to find more than one berm located at different levels (Komar, 1998); a beach scarp may be found between the berms; this is a nearly vertical face cut into the beach profile by wave impacts at the top of the beach face; however, it is possible to find scarps formed in the past what show evidence of further erosive episodes.

1.4 Characterization of mixed beaches

Given the increasing pressures to the coastal environment by the marine activities in New Zealand where the presence of mixed beaches is common, Kirk (1980) carried out a study on mixed beaches in order to summarise a general description of their morphodynamics, sediment and physical processes characteristics that govern the beach changes. In general, the mixed beaches studied present an erosional appearance forming a narrow high-energy shore zone, where the coarse sediments lay landward over a broad flat terrace of sand that extends seawards. The different texture distribution between the beach deposits and the inner shelf deposits, predominant coarser and finer materials respectively, suggested that the sediments behave as two different systems according to the size range. Also, there is no periodic cross-shore transfer of sediments, onshore-offshore between the sub-aerial beach face and the seabed in the nearshore area, characteristic of pure sand and gravel beaches (Kirk, 1980). As a result, Kirk described the typical mixed beach with a broad and planar upper foreshore; a distinct narrow high energy shore zone, consequently, a steep foreshore beach slope, with an active beach face slope between 5º and 12º, that results in a very strong reflective beach against wave energy. The foreshore is dominated by the swash and backwash processes, being classified as run-up dominated environments. Another distinct element identified at the lower foreshore was the break-point step or low-tide terrace; according to Kirk (1980), it is characterised by coarse sediments and it is the point at which waves break at all stages of the tide. Finally, Kirk described as a distinctive element characteristic of mixed beaches, a narrow and very steep nearshore face just beyond the break-point, which leads to a gentle slope of fine sand sediments. According to the characteristics described above, the author defined the always controversial seaward limit of the beach as the boundary close to the shore marked by the sudden changes in sediment texture and morphology. Regarding the interactions between sediments and beach morphology, it is concluded that the foreshore slope is intrinsically controlled by the sediment size and sorting through the permeability.

In agreement with Kirk (1980), Nicholls and Wright (1991) noted that the sediment dynamics on mixed sand and shingle beaches are clearly much more complex than for sandy beaches. They pointed out that normally those beaches present a high tide shingle and low tide sand beach where the proportion of longshore wave energy available for shingle movement will fluctuate with the tidal elevation being a maximum at high water.

Davidson et al. (1993) noticed the relevance of the tidal variation in the coefficient of reflection on mixed beaches, with the tidal stage the slope varies so it seems that a higher reflection coefficients correspond with steeper shorefaces (López de San Román-Blanco, 2003). Further investigations about the influence of the tide on the beach morphodynamics have suggested that the wave height at breaking is modulated by the tide (Davidson et al., 2009). This causes the migration of the break-point step landwards that in turn, will mobilise the sediments across the lower beach (Ivamy and Kench, 2006).

In 1997, Mason et al. noted that mixed beaches may be classified into two categories: the first, a homogeneous mixture of both sand and shingle with a grading cross-shore and longshore where the sandy fraction seaward is exposed during the low spring tides; the second, a composite beach where a sandy inter-tidal terrace is protected by a shingle formation. In a study to compare the hydrodynamics and sediment transport on composite (mixed sand and shingle) and pure sand beaches, it was noted that the reflection of swell waves by the mixed, shingle and sand, section, increased proportionally with the beach gradient above 0.06, i.e. 3.5º; however, any relationship was observed for gradients lower than 0.06. The authors also noted that the sediment composition is related to the morphodynamic response in terms of its hydraulic conductivity. It was observed that the percentage of sand controlled the hydraulic conductivity and consequently the level of energy dissipation through infiltration. Mason et al. (1997) concluded that a mixed beach will reflect more energy than both sandy and shingle beach because the dissipation of the first one due to a steeper gradient and because the infiltration on a shingle beach.

Due to the wide range of sediments which exhibit steep gradients Van Wellen et al (2000) also highlight the differences from sandy beaches regarding refraction processes and the effects of permeability. Due to the steep slope on a shingle beach the wave transformation takes place in a narrower zone than is confined in a sand beach, thus the waves approach at the beach face with some considerable angle. Indeed, shingle beaches have high hydraulic conductivity and low specific retention. These factors, as Mason et al (1997) pointed above, are important within infiltration and exfiltration processes and also to be considered for groundwater flow models. The permeability is controlled by the porosity, sediment size, sorting, grain, packing and grading and is related to the amount of free space that there is for the fluid to flow (López de San Román-Blanco, 2003). Kulkarni et al. (2004), in line with these conclusions, noted that the grain characteristics, permeability and the slope have major influence on the beach response that the effect of the tidal cycle.

Therefore, it has been noted that according with the size of material, its hydraulic properties, the permeability that controls the beach slope, and the hydrodynamic characteristics, beaches will behave as dissipative (sand) characterized by a wide surf zone and spilling lines of breaking, or reflective (mixed and gravel) characterized by a narrower surf zone with a steep beach face and normally surging or plunging waves (Van Wellen et al., 2000; López de San Román-Blanco, 2003). Mason and Coates (2001) suggested that on a mixed beach, a content greater than circa 25% of sand by weight in the sediments, within a meter from the surface, the profile response of the beach is not the same as a gravel beach.

López de San Román-Blanco (2003) state that, in general, mixed beaches in UK are characterized by a bimodal sediment size distribution, with the gravel fraction making up approximately 80% of the total. Indeed a high variability of the beach morphology has been noticed despite this consistency in the proportions of sediment fractions.

However, She et al. (2006) despite finding the degree of bimodality and the nature of the mixture a key factor for the threshold of incipient motion, suggested that the actual percentage of sand on a mixed beach is difficult to determine and likely to be variable over time. Therefore, they questioned the fact by which simply the percentage of sand in a mixture is able to indicate the performance of a mixed beach.

The study of spatial and temporal variations of sediment size on mixed sand and gravel beaches is important for beach nourishments schemes; the consideration of only one sediment size as representative of the sediment size for modelling purposes is also questionable; thus, a consistent method for sediment sampling on mixed beaches that can be representative of the system, as well as the validation of numerical models for mixed beaches to predict transport rates or profile evolution need further research (Horn and Walton, 2004).

1.5 Identification of the thesis aim

The work presented in this thesis focuses upon providing a better understanding of the longshore sediment transport and coastal processes on a mixed beach. It is suggested here that behaviour of a composite beach can be modelled effectively in the long term using a one line model approach where the model is calibrated with short term field data from an impoundment experiment. Thus the behaviour of these environments can be seen dominated by longshore processes operating on the gravel berm.

The information is aimed at providing new knowledge and approaches of interest to coastal groups and coastal local authorities for long term coastal management, to improve fundamental understanding and the body of knowledge.

To summarise, this study will:

- Contribute with a site specific data set for a mixed beach derived from a novel field experiment; the data are based on beach topographic surveys and contemporary nearshore hydrodynamic measurements collecting wave and tidal data.

- Conduct a calibration of a suitable morphological one line model for a specific mixed shingle and sand beach, widely used to estimate longshore sediment transport rates.

- Provide a new approach for long term shoreline evolution modelling applicable to mixed shingle and sand.

- Contribute to understand longshore sediment transport processes on a mixed beach and the implications of the new approach for coastal management schemes.

Chapter 2

2 Longshore sediment transport: measurements and modelling

2.1 Hydrodynamic and sediment processes in the nearshore zone

Having commented the different areas of the littoral zone and the general coastal features in Chapter 1, the next step is to describe the physical properties of the sediments which have an effect on the beach configuration and sediment transport processes. Given that the definition of beach states that it is a 'deposit of non-cohesive material (e.g. sand, gravel)' (Rogers et al., 2010), these are the relevant sediment types under consideration. The sand and gravel are traditionally classified according to the Wentworth scale that defines the sand, within a range from very fine to very coarse, between 0.0625 and 2mm; and the gravel as the material with a size larger than 2mm up to more than 256mm, also within a range of different sub-divisions (Reeve et al., 2004). The coarse sediments referred to as shingle sediments are rounded gravel and they are a common feature in an important number of beaches in the UK. The sediments play a significant role as they make up the beach and protect the coastal regions by dissipating the wave energy.

The grain size is one of the sediment properties that is considered into sediment transport formulations given its significant influence on the beach morphology. Masselink (2011) notes that the simplest calculations of a grain size are the measurements of the lengths of the longest, intermediate and shortest axes that are termed, by convention, a, b, and c axes respectively.

Statistical parameters estimated from a sediment size distribution, are good indicators to describe some sediment characteristics of a beach (SPM, 1984). Those grain size characteristics may be represented by the mean, median, sorting, skewness and kurtosis. The most widely used formulae to define those statistic measurements are presented and explained in Appendix A.4.

Some studies have suggested different transport systems when considering pure sand or gravel or mixed sand and coarse sediments. Kirk (1980) considered an activation depth for mixed sediments, and noted a no periodic onshore-offshore recirculation of sediments between sub-aerial beach face and the nearshore bed (e.g. storm/post-storm or seasonal cycles) as occur in pure sand or gravel beaches. However, Saini (2009) observed a lower depth of activation for pebbles, but once waves rework sediments the difference is little.

As the waves propagate from deep water to shallow water they experience a transformation beginning once the depth has reduced by up to approximately one half of the deep water wavelength according to the Airy Linear Theory. During this process the wavelength and velocity progressively decrease while the wave height increases and the wave period remains constant. While waves approach the coast, just beyond the breaker zone, the wave height increases up to a point at which the crest becomes over steepened and unsteady and consequently the waves break.

The wave currents as consequence of the wave breaking within the surf zone and the tidal currents enhance the sediment transport processes, especially in the swash zone where the effects of the tides enhance the action of the waves.

Masselink (1993) and Masselink et al. (1993) developed a model to simulate the effects of the tides on beach morphodynamics and demonstrated that the swash, surf and shoaling zones shifted up and down the beach with the tide. He considered the ratio between the tide range and wave height (relative tide range) as the main factor to analyse the variability of the beach morphodynamic related to the tide range. Thus, he estimated that for small relative tide range (<2m), the processes were predominant in the swash and surf zone. This research was carried out for sandy beaches in Queensland (Australia) but with different range of sediment size related to the dimensionless fall velocity given by Gourlay (1968) and it was found that an increase in sediment fall velocity the dimensionless fall velocity decreased and the beach profile became steeper and more reflective.

2.2 Methods for measure Longshore Sediment Transport

2.2.1 Field measurements

The direction and magnitude of the longshore sediment transport are important to evaluate their effect on the coast and assess likely beach erosion for the design of coastal protection structures or any other maritime engineering works. Thus there are some qualitative indicators that may provide evidences of existing sediment transport processes, and in consequence, some quantitative indicators that can be measured to provide estimations of the processes involved (CERC, 2002).

Wave and current conditions, coastal geomorphology, sediment properties as the size and composition may be significant qualitative indicators for coastal changes. Some evidence of the sediment transport direction are the accretion and erosion of sediments when significant structures as groynes act as a barrier for the littoral drift, geomorphologic changes observed as sediment depositions or displacement of the shoreline at headlands, inlets or beach alignment.

Considering the mentioned qualitative indicators, there are different methods for quantifying the sediment changes involved. Some of those methods are the topographic surveys of the beach profiles, hydrographic surveys and analysis of aerial photographs (CERC, 2002). Indeed, conducting those surveys or aerial images repeatedly over the years may give an indication of the net sediment transport rates in the long term, from 10 years.

Another methodology for estimating longshore transport rates is that based on the measurements of the sediment depositions blocked by a normal structure to the shoreline and the resulting erosion at the adjacent side of the barrier (CERC, 2002). The LST can be quantified by assuming that the LST rate is 0 at the normal structure (SPM, 1984) as no sediment passes through the structure and below its crest. In order to estimate the LST rates and potential changes due to the presence of the structure, the beach profiles can be measured using a Global Positioning System (GPS) adjacent to the structure (Van Wellen et al, 1998). Then, based on the Energy Flux method, those measurements can be related with estimations of the longshore energy wave power.

Looking at the literature there are only two applications of the short-term impoundment method in the field and for sand, those of Bodge (1987) and Bodge and Dean (1987).

Looking at short term estimations of the longshore sediment transport, another quantitative method is the use of sediment tracers (e.g. Komar, 1977; Blackley, 1980; Kraus et al., 1982; Lee et al., 2007), sediment traps (e.g. Chadwick, 1987); or optical devices such as the OBS, optical backscatter sensor, which can give estimations of the suspended sediment concentrations and turbidity.

Kraus et al. (1982) used fluorescent sand tracer experiments to measure the short-term longshore transport rate in an energetic surf zone on natural beaches and beaches near structures. Despite finding longshore transport rates measured in agreement with the predictive expression of Bagnold (1963) which includes wave power at breaking and longshore current velocities, they encountered problems related to the advection speed alongshore of the tracer which encountered limitations to the method. They conclude that there was need for a minimum of three tracers to see the distribution of sand transport rate across the surf zone.

It is seen that there are available different methods to determine longshore sediment transport rates, and the capability of all of them to provide accurate results will depend of some aspects such as: the type of beach, the beach parameters considered as indicators of the beach morphology change, and whether data are collected from field or laboratory experiments. In a review of the LST equations for gravel beaches, Van Wellen et al. (2000) point out the lack of data and information related to coarse and mixed beaches due to the limitations to deploying delicate instrumentation on shingle sea bed shores.

In general tracers and traps tend to over-estimate the LST and present limitations regarding to the shingle sediment size and it is associated with more variability in space and time rather sandy beaches. The application of tracers on shingle beaches in general has the restriction of the depth of moving layer for sediments that is deeper than a sand sea bed. Volumes obtained from traps measurements have been found lower than those measured with tracers (Bray et al. 1996 in Van Wellen et al. 2000) and this is related to the tidal oscillations overall in macro tidal environments where the sediments are moved considerably onshore and offshore.

A review of field data suitable to estimate LST rates carried out by Schoonees and Theron (1993) relates that only two experiments of 42 listed were performed on coarse grain beaches, those that Nichols and Webber (1987) and Nichols and Wright (1991) reported on Hurst Castle Spit (in 1981 and 1982) and near Hengistbury Head respectively and that one accomplished by Chadwick (1989) on Shoreham Beach.

2.2.2 Numerical modelling

2.2.2.1 Numerical model fundamentals

Numerical or computer modelling is one of the methods most used to test the applicability of the problems within the coastal engineering field. Its use is justified since it is a tool rather than physical models and in general they are more accessible at the time to work with them.

To formulate a numerical model the equations that govern the processes to be modelled must be previously well understood. In order to set up the model, the numerical methods, transfer functions and calibration coefficients must be known also. Also, depending on the space-time scale to be studied, models are classified as S, M or L models, short, medium and long term respectively (Kamphuis 2000). Due to the increasing demand of predictive tools in long-term during the last decade by the coastal authorities, researchers are looking at the concept of L-models with the purpose to predict shoreline changes in 70 or 100 years.

The widely known one-line models have practical capability and have been demonstrated to predict shoreline change in the long term (Dabees et al., 1998).

Field data, as well as data measured in laboratory experiments, are normally used to test numerical models to validate their applicability, simply varying the value of the different variables.

2.2.2.2 One-line models

The one-line model, or one-dimensional, is the simplest predictive tool since it considers all the contour lines to have the same shape and move together as if they were only one contour line to describe the whole beach movement seaward and landward (Kamphuis 2000). Of one-dimensional models, the most known are ONELINE model (Kamphuis, 1993; Dabees and Kamphuis, 1998, 1999; Dabees, 2000) and GENESIS

(Hanson, 1987; Hanson and Kraus, 1989; Gravens, Kraus and Hanson, 1991), in Kamphuis (2000). Overall, one-line models consist of solving two one-dimensional equations that are an equation to estimate the LST rate integrating waves and beach parameters and the equation of conservation of mass (Kamphuis, 2000).

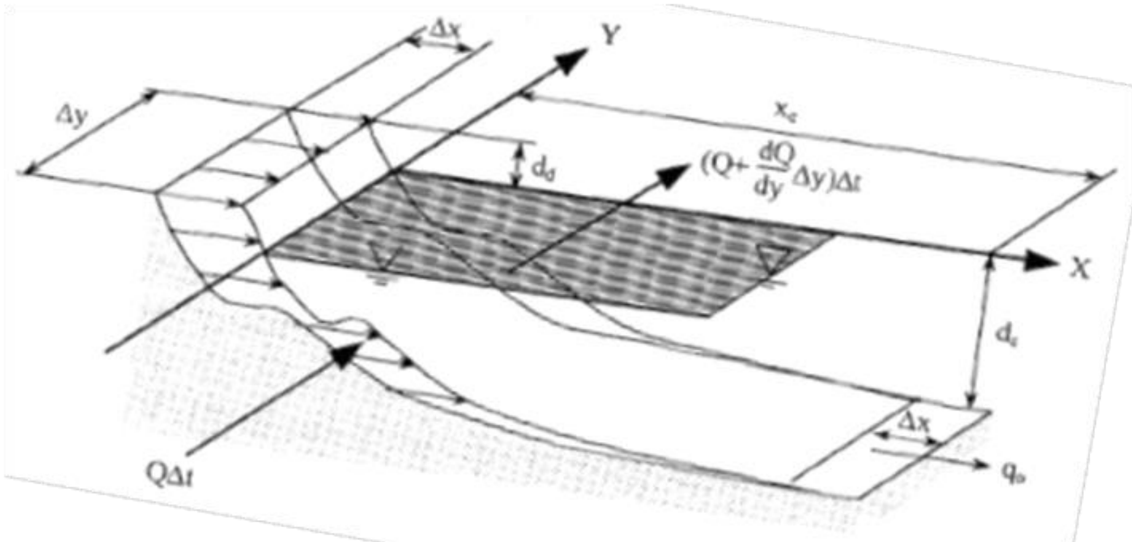


Figure 2.1 One-line model scheme based on the Conservation of Mass Equation (image courtesy of *Introduction to coastal engineering and management. Advance Series on Ocean Engineering Vol.16*, Kamphuis, J.W. Copyright@2000 World Scientific).

Looking at Figure 2.1, d_c is the closure depth, as it is known, it is the depth at which is considered that beach profiles are not affected by the normal incidence of wave conditions and it can be measured by topographic or hydrographic surveys (Kamphuis, 2000). Thus, the one line model concept considers the beach profile with a constant shape that slides along a horizontal plane located at that closure depth. Alongshore direction is given by the y-axis and the x-axis indicates the distance to the shoreline with respect to the y-axis. Overall, assuming that the profile remains the same, the one line equation indicates that all contours move the same distance in a way that the whole beach movement can be represent by only one contour line (Kamphuis, 2000).

In general both one-line models are based on the assumption that the shoreline moves as one contour parallel to itself out to a limit defined by the depth of closure and this contour conforms the beach profile change within the defined boundary conditions under the assumption of the conservation of sediment for a considered length of the coastline.

GENESIS estimates the shoreline changes produced by the spatial and temporal differences in the longshore sediment transport due to the breaking waves (Gravens et al., 1991; CEM, 2002). Considering a grid of cells in a coastline, GENESIS computes the shoreline change in time as a function of the breaking wave height and angle (CEM, 2002). However, the ONLINE model introduces the shoreline change due to the cross-shore transport and it does not consider the shoreline direction and incident wave angle (Dabees et al., 1998).

Some authors considered also the importance of the cross-shore changes or other parameters involved that were not taken into account in the one-line approach. In order to understand the beach profile behaviour for the shingle beach, Powell (1990) developed a model to predict the short term response for shingle beaches. For considering the analysis of the profile in short term, the model helps to extrapolate the short term to long term relating the those changes to the shoreline behaviour in a qualitative way as the shoreline is understood in long term and using LST data for modelling. Brampton and Goldberg (1991) noted the importance of considering 3D modelling to include effects of other parameters on the cross-shore dimension such as the scour due to the interaction with structures.

2.2.2.3 Longshore sediment transport formulae

There is a categorisation of the existing longshore sediment transport equations depending on the method applied, in general those equations are divided as follow (CERC, 2002): energetic methods, such as the Energy Flux method that has been developed specifically for coastal applications; the stream power approach (Bagnold, 1963, 1966,1980,1986; Bailard, 1981,1984; McDowell, 1989, and Chadwick, 1991) that is more generally applicable in the coastal area; then, the force-balance method and finally, the dimensional analysis method.

The volumetric transport rate denoted as Ql, with units of cubic meters per day, is the form of the longshore sediment transport rate in engineering applications (CERC, 2002). It is defined as the total volume that would be measured by survey of an impoundment at a groyne. Also, the LST rate Q_l, may be expressed in terms of the immersed weight transport rate I_l that is described in detail in Section 2.2.1. The latter presents some advantages respected to the volumetric transport rate that are explained later.

Bayram et al. (2001) evaluated the predictive capability of six well known sediment transport formulae and adapted to calculate the cross shore distribution of the LST rate. They noted that different formulae responded differently for the same data sets and highlight the importance of field data for calibration purposes and obtain values that can be apply to a wider range of wave and beach conditions.

2.3 The Energy Flux Method

2.3.1 CERC Equation

In 1950, the US Army Corps of Engineers (USACE) applied for the first time in the United States the Energy Flux Method using a formula to predict the longshore sediment transport based on the wave energy (CERC, 2002).

The CERC (2002) summarizes the evolution of the named CERC Equation based on the application of the Energy Flux method. Thus, the earliest documented measurements of the LST rate related to wave energy were by Watts (1953a) and Caldwell (1956). Later on, in 1962, Savage developed an equation using a summary of the available field and laboratory data; in 1966, that equation was adopted by the US Army Corps of Engineers as part of the coastal design manual when it became known as the CERC Equation.

That CERC formula was given as a volumetric rate, and Inman and Bagnold in 1963 suggested to apply an immersed weight longshore transport rate. Later on, in 1970, Komar and Inman calibrated the immersed weight sediment transport equation using available field data and their tracer-based measurements at Silver Strand in California and El Moreno in Mexico. Finally, in 1966, the CERC Equation version for littoral sand transport was updated based on the transport relationship developed by Komar and Inman in 1970 and other available field data and it was presented in the editions of the Shore Protection Manual in 1977 and 1984 (CERC, 2002).

Basically, the Energy Flux method considers the immersed weight of the alongshore moving sediment proportional to the alongshore wave power per unit length of beach (Kamphuis et al. 1986).

Then, based on the Energy Flux method, one of the most common formula applied in coastal engineering as a tool to estimate sediment transport rates for beach management problems is the CERC equation (SPM 1984) and originally, it has been developed to calculate the longshore sediment transport rates for sandy beaches in which, the k is the transport rate coefficient. This empirical formula is represented by Eq. 2.1:

$$I_l = kP_l \qquad \text{Eq. 2.1}$$

WhereI_l is the immersed longshore transport rate, k is the sediment transport coefficient and P_l is the longshore wave power. The immersed weight longshore transport rate should have same units as the alongshore wave power, e.g. N/sec. The equation that relates the volumetric transport rates Q and the immersed weight rate I_l is the relationship given by the Eq. 2.2:

$$I_l = (\rho_s - \rho)ga'Q \qquad \text{Eq. 2.2}$$

Where ρ_s and ρ are the density of the sediments and sea water respectively, g is the acceleration of gravity, a' is the relation of volume solids between the total volume and Q is the empirical LST rate (or volumetric transport rates).

In Eq. 2.1, Pl is the alongshore component of wave power in the breaking zone what should be indicated by the subscript b, thus it is defined and written in the form, Eq. 2.3:

$$P_{l_b} = C_{g_b} E_b \cos\alpha_b \sin\alpha_b \qquad \text{Eq. 2.3}$$

In Eq. 2.3 C_{gb} and E_b are the wave group velocity in shallow waters and the wave energy at breaking respectively, and they are expressed by Eq. 2.4 and Eq. 2.5 α_b is the wave angle direction.

$$C_{g_b} = \sqrt{gh} \qquad \text{Eq. 2.4}$$

$$E = \frac{1}{8}\rho g H^2 \qquad \text{Eq. 2.5}$$

Where g is the gravity acceleration and h is the water depth. H is the wave height.

Therefore, the alongshore wave power can be written in the form, Eq. 2.6:

$$P_{l_b} = \frac{1}{16}\rho g H_{s_b}^2 C_b \sin 2\alpha_b \qquad \text{Eq. 2.6}$$

The use of the immersed weight transport rate rather than the volumetric transport rate presents two clear advantages. One it is that the immersed weight transport rate incorporates the effects of the density of the sediment grains; and secondly, the empirical transport rate coefficient k becomes dimensionless.

Another significant aspect to take into account is the definition of the type of wave height at breaking that is used to calculate the longshore wave power. There is an important difference in the longshore transport rate coefficients whether the root mean square wave height at breaking $H_{b_{rms}}$ is used or it is the significant wave height at breakingH_{b_s}. In general, the estimated sediment transport coefficients using $H_{b_{rms}}$ are larger than those obtained usingH_{b_s}. A summary of different documented *k* coefficients is presented in Table 2.1and Table 2.2in Section 2.3.3.

2.3.2 Development of the CERC Equation (1984)

Further investigations to attempt reliable estimations of the LST rate applying the CERC formula for different locations identified the need for further modification to the equation and to consider other parameters that affect sediment transport processes that had not been taken in account. Thus, based upon laboratory and field data, Kamphuis (1986) proposed another equation that considers the beach slope, m, and sediment size, D, see Eq. 2.7, (Kamphuis et al.1986). The subscripts s and b refer to significant wave and in the breaking zone respectively.

$$Q_s = 1.28 \frac{mH_{sb}^{7/2}}{D} sin\, 2\alpha_{sb} \qquad \text{Eq.2.1}$$

Then, Kamphuis (1991) developed a new expression, Eq. 2.8, that involves the effects of the peak wave period, T_p, beach slope m and sediment size *D*. This formula had been developed by Kamphuis (1991) and validated with laboratory data. Later on, it was validated also with field data and its form is as follow (Kamphuis 2002):

$$Q_s = 2.27 H_{sb}^2 T_p^{1.5} m_b^{0.75} D^{-0.25} sin^{0.6} 2\alpha_b \qquad \text{Eq.2.2}$$

Bayram et al. (2007) considered any other processes that have influence in the sediment transport such as wind, tide and breaking wave to develop a new formula, Eq. 2.9, has been validated with an extensive data set including a wide range of wave, sediment and beach conditions in order to provide a reliable accuracy to the predictions.

$$Q_{lst} = \frac{\varepsilon}{(\rho_s - \rho)(1-a) g w_s} F\bar{V} \qquad \text{Eq.2.3}$$

Whereas CERC equation (CERC, 1984) and Kamphuis (1986 and 1991) consider only wave-generated currents as the majority of the LST formulae, the Bayram equation (2007) looks at the flux of wave energy shoreward throughout the term F and also the term $\overline{V}$ includes the longshore currents involved. The parameter ε is the transport coefficient and represents the efficiency of the waves in keeping sand grains in suspension (Bayram et al., 2007).

Eq. 2.10 represents the formula proposed by Van Wellen et al. (2000) developed for shingle beaches which estimates LST rate in terms of (m^3/s) and the coefficients c0, c1, c2, c3, c4 and c5 are determined by fitting the equation into a data set and applying an iterative procedure until obtain the values that best match with those obtain with the model, in this case BORESED model (Chadwick, 1991).

$$Q = c0\frac{(1+e)}{(\rho-\rho_s)}H_{sb}^{c1}T_z^{c2}tan\alpha^{c3}D_{50}^{c4}sin2\theta_b^{c5} \qquad \text{Eq.2.4}$$

As conclusion, Van Wellen et al. (2000) found out a good correlation between estimations obtained with the new formula and those from the model, as they were calibrated for the same field site, Shoreham beach. Thus, further calibration is required for different locations in order to demonstrate the capability of the formula to predict LST rates in a shingle beach. Also, in the CERC Equation, it is suggested that the k coefficient is a function of the statistical wave height that it is used in the equation (Van Wellen, 1999).

In general those empirical formulations are tested using field data in order to calibrate and validate the applicability of the longshore sediment transport formulae. In this study, data collected during a field campaign based on an experiment will be used to calibrate the CERC Equation and afterwards, as potential future work, it would be interesting use the data for calibration of other well-known formulations for the sediment transport such as Kamphuis (1991, 2002), Bayram et al (2007) and Van Wellen et al. (2000).

2.3.3 Empirical sediment transport coefficient k

According to the CERC Equation (SPM, 1984), it is seen that the k parameter is an empirical coefficient of proportionality for the longshore sediment transport. This coefficient may be dimensionless applying the immersed weight transport rate equation, Eq. 2.1. It has determined from different studies that the range of variability of k values depends on whether the longshore wave energy is calculated using $H_{b_{rms}}$, root mean square wave height, or H_{b_s}, significant wave height, in both situations at the breaking line; unfortunately, there are some cases where the type of wave height used is not specified in the literature. The use of $H_{b_{rms}}$ gives larger values of k than those using H_{b_s}, being the conversion factor almost two (Nicholls and Wright, 1991).

There are few studies that attempted to apply the CERC formula on gravel beaches, as the CERC formula was originally developed for sand.Table 2.2summarizes the k coefficient values documented in the literature that may be relevant for comparison with the results presented in this study. It is observed that mainly empirical dimensionless k values for shingle beaches are around 0.01 (Wright, 1982; Bray, 1990, Kos'Yan 1994 and Voulgaris et al. 1999).

The Engineering Manual (2002) summarizes a comparison between different field data sets obtained applying different techniques for measuring LST rates for sand. Those data are related to the immersed weight transport rate (N/sec) and the longshore wave energy (N/sec) using H_{rms_b}. The techniques followed consisted on measurements of sand deposition at jetties and breakwaters, sand tracers and sediment traps. While the SPM (1984) presented a k coefficient of 0.39 based on computations using $H_{b_s} k_{SPM_s} = 0.39$), the calibration of the Engineering Manual (2002) using the field data and the $H_{b_{rms}}$ presented a k coefficient of 0.92 ($k_{SPM_{rms}} = 0.92$) (CERC, 2002). Also, it is documented a k coefficient of 0.77 obtained by Komar and Inman (1970) using $H_{b_{rms}}$. ($k_{K\&I_{rms}} = 0.77$).

The Engineering Manual (2002) also reports that there are other studies that have related the variation of the k coefficient with the median grain size and the surf similarity parameter known as the Iribarren number.

In general, there was observed a decreasing trend of empirical *k* coefficient values being smaller for shingle beaches in comparison with those obtained for sandy beaches (CERC, 2002), thus, the use of a *k* coefficient for sand on shingle beaches may over-predict the longshore transport rate (Nicholls and Wright, 1991). As Komar (1988) pointed out, the quality of the data is likely to have an effect on the correlation analysis between the *k* parameter and the sediment size in order to obtain the dependency trend between both.

Respect to the Iribarren number, Eq. 2.11, Kamphuis and Readshaw (1978) observed from laboratory data that the *k* coefficient increases with increasing the value of the surf similarity parameter (CERC, 2002) that may be an indication of the rate of dissipation of wave energy (Kamphuis, 1986).

$$\boldsymbol{\xi_b = \frac{m}{(H_b/L_0)^{1/2}}} \qquad \text{Eq. 2.5}$$

Eq. 2.11 represents the ratio between the beach slope, *m*, and the wave steepness given by H_b/L_0. This relation indicates the breaker type and it is also known as the surf similarity parameter ξ_b (Kamphuis, 2000). Battjes (1974) established that $\xi_b < 0.4$ corresponds with spilling breaker waves and are identified with flat slope beaches that are mainly characterized by sand sediment size and are considered dissipative beaches. However, values between the range $0.4 \leq \xi_b \leq 2.0$ corresponds with plunging breaker waves and those $\xi_b > 2.0$ to collapsing breaker waves. The latter are identified to take place in steep beaches that are characterized for presenting coarser grain sediments; also surging breaking type occurs in very steep beaches.

A study by Nicholls and Wright (1991) describes the use of three experimental data sets for determining the *k* coefficient for pebbles using aluminium tracers The experiments were conducted at two different shingle beaches sited in Christchurch Bay (South UK), at Hengistbury Long Beach and at Hurst Castle Spit, the latter is located at the eastern end of Milford-On-Sea, the site of this study. According to the authors, Hurst Spit mainly is formed by shingle sediments that form the bulk of sediment and occupies the entire active profile, including offshore zone. Thus, they considered a unique transport rate because under even moderate energy conditions the sand moves offshore leaving essentially a mobile layer of shingle. Consequently, for Hurst Spit they assumed that any energy used to move sand must be negligible.

Table 2.1 Summary of documented empirical *k* coefficient values for sand sediment beaches.

k coefficient	Author	Method	Sediment Size	H (m) /wave conditions	Reference
0.77	Komar and Inman, 1970	Field data, tracer experiments. Dimensionless	Sand	$H_{b_{rms}}$	Komar and Inman, 1970
0.39	SPM 1984	Based on computations. Dimensionless	Sand	H_{b_s}	SMP 1984
1.28	Kamphuis et al., 1986	Units: [M/L5/2T].Proposed new expression for k	Sand	H_{b_s}	Kamphuis et al. 1986
0.29	Komar 1989	Energy Flux. Dimensionless	Sand	H_{b_s}	In Nichols & Wright 1991
0.92	EM 2002	Energy Flux. Field data. Dimensionless	Sand	$H_{b_{rms}}$	EM 2002

Table 2.2 Summary of documented empirical *k* coefficient values for coarse sediment beaches.

k coefficient	Author	Method	Sediment Size	H (m) /wave conditions	Reference
0.0025	Hattori & Suzuki, 1978	Field data, tracer experiments. CERC (1984)	Shingle		Hattori & Suzuki 1978
0.023	Nicholls, 1982	Field data, tracer experiments.	Shingle		Voulgaris et al., 1999
0.013	Wright, 1982	Field data, tracer experiments.	Shingle		Voulgaris et al., 1999
0.002	Brampton &Motyka, 1987		Shingle		Voulgaris et al., 1994a
0.031	Chadwick, 1989	Field data, sediment traps.	Shingle	$H_{b_{rms}}$	Chadwick, 1989
0.010, storm 0.014, swell	Bray, 1990	Energy Flux. Field Data.	Shingle	Storm Swell	Voulgaris et al., 1999
0.018	Bray, 1990	Energy Flux. Field Data.	Shingle		Voulgaris et al., 1999
$0.0029 \leq k \leq 0.058$	Nicholls & Wright, 1991	Field data, tracer experiments.	Pebbles	H_{b_s}	Nichols &Wright 1991
0.012	Kos'Yan, 1994	Field data, historical bathymetric surveys.	Shingle		Voulgaris et al., 1999
0.01	Schoonees& Theron, 1994	Author's note: use only for order of magnitude.	Coarse	H_{b_s}	Schoonees& Theron, 1994
0.04 0.20 0.36	Bray et al. 1996	Field data, tracer experiments.	Shingle	Low Intermediate High	Voulgaris et al., 1999
0.015 (mean value)	Voulgaris et al., 1999	Field data, SGN acoustic technic.	Shingle	H_{b_s}	Voulgaris et al., 1999

For the three cases they applied the same method as Komar and Inman (1970) with the slight difference that the net displacement of the tracer centroid was measured after several tidal cycles and not hours. They used the significant breaking wave height and the *k* values obtained were 7 to 100 times lower than on sand beaches, it is within a range of 1-20% of k_{Komar} = 0.29 (using $H_{b_s}$by Komar, 1989), that means a *k* value for shingle sediments between 0.0029 and 0.058 (Nicholls and Wright, 1991).

Consequently, despite of having encountered some limitations, the authors concluded suggesting the aluminium tracers as an effective technique to estimate longshore transport rates on shingle beaches and their results are in agreement with those used for modelling shingle beaches that are documented within a range of 1 and 15% of those calculated for sand.

As expected, the results of *k* obtained for Hurst Castle Spit suggested to the authors that *k* increases with decreasing grain size; the factor of differential longshore transport may be of importance. Also they observed that Hurst Castle Spit showed natural longshore transport grading with a down-drift increase in size as well as some high energy events that only caused cross-shore transport. They pointed out that the experiments were not specifically designed to estimate the *k* parameter; however, the values obtained are in agreement with other values documented in the literature. They pointed out the importance of matching the type of tracer with the indigenous grading. Also they suggest that further experiments, either using tracers or other methodologies as the use of impoundment techniques to interrupt the LST, would contribute to understand better the relation between the longshore transport and the sediment size.

Voulgaris et al. (1999), in order to compare the results obtained from an experimental study, summarized previous works in which different sediment transport coefficients were found using different techniques. Kamphuis (1991) developed an empirical relationship based mainly on laboratory results that correlates well with laboratory field data (CERC, 2002). Despite the fact that the CERC Equation has being subject to different attempts for estimating reliable estimations of LST rates for different locations, and modifications including other parameters that affect the sediment transport processes, this formula is preferred for application to field studies over other existing formulae, because it is a empirically derived relationship and based only on field data.

2.4 Discussion and conclusion

Van Wellen et al. (2000) result that volume changes estimated from beach profile survey data are the most used method to assess the beach response in the long term, also from the engineering point of view it seems to be a reliable technique whether a shore-normal structure acting as a barrier traps the longshore sediment transport assuming then, that no sediments pass through the structure and the profile change is due to the longshore transport. Supporting that, Nichols and Wright (1991) noticed that loss of shingle seaward is generally minimal, thus this technique is suitable for estimating LST rates on mixed and gravel beaches and also for calibration purposes.

Among the different methods to study beach morphodynamics, beach profile analysis is considered a good method to estimate shoreline changes in short term in relation with water level changes and wave conditions.

Between the different methods for measuring LST rates, the consideration of the impoundment concept for measuring LST rates would contribute to understand the changes that arise from the introduction of coastal defences (Brampton 1991).

Brampton (1991) noted that given the difficulty of modelling both cross- shore and longshore change in a gravel composite beach due to the transition in slope on the profile section, from an engineering point of view it is considered that a desirable shingle fraction will maintain the sand foreshore healthy too. Therefore, it would be interesting to investigate if different sediment types i.e. sand and shingle, in a mixed beach respond differently to the hydrodynamic forces and consequently behave in the same way under the sediment transport processes.

The demonstration that a regular monitoring routine is a significant part of the best practice for coastal managers, to understand the natural variability of the littoral communities. As noted by Baylard et al. (2001) field data are important for calibration purposes as it will improve the predictive capability of the formulae.

This study will assess the shoreline variability using a one line numerical model. It is the simplest numerical model for coastal morphology because this one dimensional model relies on LST rates that are indicative of the sediment movement along the coast, due to mainly the interaction of the sediments with the wave action. The LST rates will be estimated according to the Energy Flux Method by using the CERC Equation (1984). This equation will be used given the following reasons:

- It is based on the Energy Flux Method and it was developed specifically for coastal applications (CERC, 2002).
- Estimates longshore transport rates related to the longshore wave energy.
- Immersed weight rate presents two advantages over the volumetric transport rates: is dimensionless and incorporates the effects of the density grains.
- It is an empirical based equation, which implies the need of field data, and consequently, a specific site election.

The LST is subject of study to assess shoreline evolution in long term, therefore, nowadays, there is a demand by coastal authorities for developing new tools for management purposes to face the risk of flooding, especially in the areas more vulnerable to the effects of the sea level rise and climate change.

Chapter 3

3 A Novel technique for measurement of LST rates on a mixed beach

3.1 Introduction

This chapter presents a description of the methodology applied in this work for achieving a better understanding of the longshore sediment transport processes on a mixed beach. Under consideration of the previous discussion presented in Chapter 2, this study has adopted an empirical approach based on field measurements which were made during a short term experiment carried out on the mixed beach site introduced in Section 3.2. This experiment, presented in Section 3.4.1., has been integrated within the 'Field monitoring, data assimilation and Analysis' activity of the Risk-based Framework for Predicting Long-term Beach Evolution project (RF-PeBLE), financially supported by the Engineering and Physical Sciences Research Council (EPSRC) through grant reference EP/C005392/1. Then, Section 3.3.introduces the RF-PeBLE project and provides an overview of the data collection strategy for that project. The methodology used for the collection of the short term (several months) data set that is the focus of the current research is detailed in Section 3.4. Additional information related to the methodology is presented in Appendix A.

The data analysis and results presented in the following chapters of this thesis follow on from the short term experiment, which focussed on the use of an impoundment technique for measuring LST rates. Beach RTK- DGPS survey measurements, simultaneous measurement of wave and tide data, and the collection of remote video imagery from an Argus Beach Monitoring System have added to the completeness of the dataset.

3.2 Site of the study

3.2.1 Milford-on-Sea mixed beach

The site of the study is the mixed beach under Hordle Cliff, near Milford-on-Sea, located in the county of Hampshire on the Southern coast of the UK. It is a natural mixed (shingle and sand) beach sited at the eastern side of Christchurch Bay, Figure 3.1.

The bearing of the shoreline at Milford-on-Sea is 108º with respect to the geographic North.

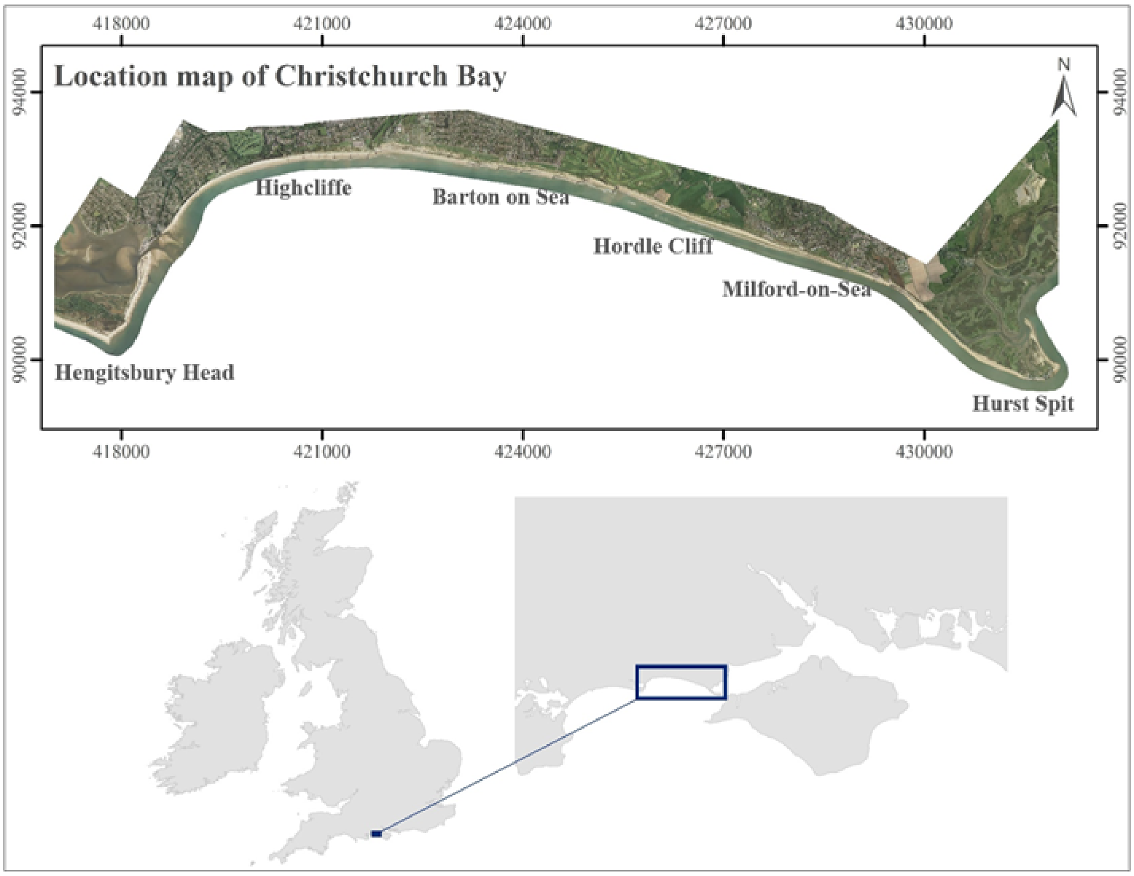


Figure 3.1 Location map of Milford-on-Sea at Christchurch Bay.

The beach at Milford-On-Sea is subject to prevailing SSW waves; the tidal regimen is semi-diurnal and mesotidal with a spring tidal range of 2m ODN (Ordnance Survey Newlyn) and a neap tidal range of 0.9m ODN (Admiralty Tide Tables, 2007). The double tidal cycle in Christchurch Bay creates a marked double high water and is associated with significant tidal currents that enhance the potential for coarse sediment transport (SCOPAC, 2004).

This length of coast is within an area of significant environmental value. The coastline from Barton on Sea to Hurst Spit is designated a Site of Special Scientific Interest (SSSI) and assessed as Jurassic Coast World Heritage Site; indeed, Milford-on-Sea is a Coastal Area of Outstanding Natural Beauty (AONB). In terms of coastal planning, Milford is subject to New Forest District Council and is covered by the Solent Strategic Guidance Plans and Local Authority Coastal Management Plans.

In addition, the interest of this area lies on the typical coastal elements presented: cliff eroding at the western side near Barton on Sea,Figure 3.2; natural mixed beach at Hordle Cliff and the western margin of Milford-on-Sea, Figure 3.3; and coastal defence structures comprising timber groynes and seawall between Milford-on-Sea and Hurst Spit, Figure 3.4.

Figure 3.2 Eroding cliffs at Barton on Sea. The sign pole marked by the circle at the top of the cliff (16th July 2007), is the same sign laying on the backshore on the right hand side (picture taken the 5th April 2008).

Figure 3.3 Natural mixed beach at Milford-on-Sea facing to west at Hordle Cliff.

Figure 3.4 Seawall and field of timber groynes at Milford-on-Sea.

Offshore of Christchurch Bay and covering an extension within the Territorial Waters Limit and beyond, there is the 'Round 3 Wind Farm' Zone (Zone 7) (The Crown Estate, 2013), at which there is a 'Round 3 Agreement for Lease' subject to approve the consent for the construction of Navitus Bay Offshore Wind Farm to be developed by Eneco Round 3 Development Ltd. Also, according to the information provided by the The Crown Estate (TCE), at the western side of Island of Wight there are two active dredging zones, two licensed dredging zones and eight aggregate dredging application areas.

3.2.2 Critique of SCOPAC report (2004)

The construction of sea defences at Milford-on-Sea dated from 1936. Nicholls et al. (1987) have commented that in the subsequent period up to 1968 there was evidence that their presence had modified the sediment budget towards Hurst Castle Spit. The SCOPAC (2004) study noted that Milford-on-Sea has been subject of periodic renourishment since 1970, however, the detailed information of the quantities as well as the location of the renourishment is not provided questioning its reliability. Indeed New

Forest District Council has confirmed there are no records of beach renourishment at Milford-on-Sea (personal communication, August 2013).

This sediment transport study reveals the relatively recent geological origin of Christchurch Bay. Its configuration has been formed by the coastline retreat during the mid to late Holocene transgression in the Quaternary Period. This study reports a general historical retreat of the cliffs even with some evidence of historical rock fall and relict landslide recorded. In particular it pointed out that the simple landscape of Milford-on-Sea is in progressive erosion and susceptible to cliffs landslide, whereas Hordle Cliff is subject of periodic coastal erosion hazards. Thus, the impact of climate change and the damage of coastal defences at this area have requested more consideration to be appraised by the coastal management plan. The seabed sediments offshore of Milford-on-Sea are classified as sandy gravel according to the sediment classification modified after Folk, 1954, (Hamblin et al., 1992).

The study (SCOPAC, 2004) also summarizes previous coastal evolution works; it has noted that in Christchurch Bay the bedload transport system is closed since the late Holocene. However, human activity has caused some changes to the sediment budget, e.g. marine dredging activity, soft engineering schemes and coastal defences. Still, despite of some minimal inputs and losses, according to Velegrakis (1994) the overall system is considered quasi-isolated (op cite SCOPAC, 2004). This study summarises the gravel, sand and clay sediment inputs observed from cliff or coastal slope erosion at both sides of Hordle Cliff, see Figure 3.5. According to the SCOPAC map above, a predominant littoral drift of sand and gravel has been observed in Christchurch Bay taking place from West to East. That littoral drift towards Hordle Cliff has been estimated being about $10000m^3a^{-1}$ and diminishes to $3000\text{-}10000m^3a^{-1}$ from there to Milford-on-Sea. Then, from Milford to Hurst Spit the littoral drift quantified increases up to more than $20000m^3a^{-1}$. At Hordle Cliff a sand motion onshore-offshore has been noticed however there is no quantitative data of the sediment flow volume. It is also observed an offshore sediment transport of sand from Dolphin Bank to Dolphin Sands; at the East margin of Christchurch Bay, in the Needles Channel, the sediment source called Shingles Bank is found. At that margin of the bay take place two sediment transport mechanisms: an offshore sediment transport of gravel towards Hurst Spit from the Shingles Bank; an Offshore Estuarine transport of gravel and sand seawards along the Needles Channel.

The observations above suggest that these sediment transport patterns and estimations should be considered more qualitative rather than quantitative as the study does not provide detailed information of the methods used for estimating the sediment transport rates as well as e.g. the sediment rates supplies by cliffs and coastal slope erosion.

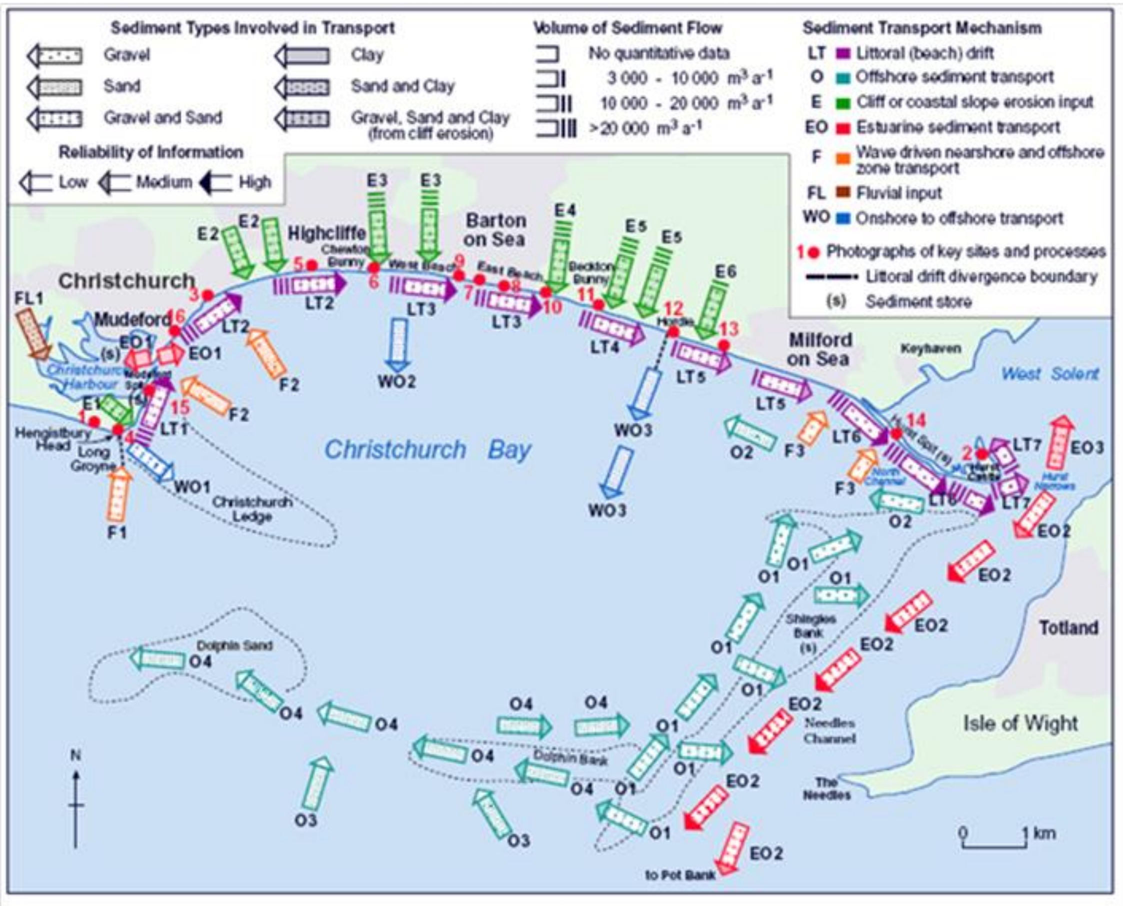


Figure 3.5 Sediment transport patterns at Christchurch Bay, from Hengistbury Head to Hurst Spit (image courtesy of SCOPAC, copyright©SCOPAC MMIV).

3.2.3 Milford-on-Sea in the context of the Shoreline Management Plans

The Bournemouth Borough Council, Christchurch Borough Council and the New Forest District Council are responsible of the administration of the Christchurch Bay CBY process unit. The local SMP 1 for this length of coast was commissioned by the Poole and Christchurch Bays Coastal Group and produce by Halcrow (now CH2M HILL) in 1999, (EA website, 2011). Later on, between 2008 and 2010, the review of this SMP 1 concluded with the publication of the second generation of the SMP, known, in general as the SMP 2. In this occasion, the review was commissioned also by the Poole and Christchurch Bays Coastal Group but carried out by Royal Haskoning UK; it was lead by the Bournemouth Borough Council and supported by the Environmental Agency. The final document was published in October 2011.

The Poole and Christchurch Bay Shoreline Management Plans are subject to the Subcell 5F that extends from Durlston Head, at the South of Christchurch Harbour, to Hurst Spit at the eastern side of the bay; and the SMP 2 considers four Policy Development Zones. The shorelines of Hordle Cliff and Milford-on-Sea are part of the Christchurch Bay (CBY) Process Unit within the PDZ 1 which covers the central and eastern side of Christchurch Bay. The relation between the locations and policies considered in the SMP1 and SMP2 is presented in Table A.1 (Appendix A) this table summarises the policies adopted by both SMPs. According to the SMP2, the preferred management policy has been assessed to be implemented in long-term, this is by considering three future epochs, to be specific: 2025, 2055 and 2105.

In addition to the SMPs, the mixed beaches at Milford-on-Sea and Hurst Spit have been subject of a long term beach management programme being monitored since 1989 (Bradbury et al., 2003). Bradbury et al. (2003) analysed the beach morphodynamic response using beach profile survey data, in order to assess the potential impacts on the shoreline due to the aggregate dredging activity in the Shingles Banks. The control site analysed at Milford-on-Sea has shown an eroded beach profile trend previous to the dredging activity which continued afterwards. However, the authors, could not provide a firm conclusion as to whether that erosive trend could be clearly related to the aggregate extractions due to the lack of post-dredging data at these sites. Therefore, Bradbury et al. (2003) noted the need for the establishment of site specific baselines, and the importance of the strategic monitoring survey programmes in the UK. As a conclusion, further research is needed to characterise coastal environments and continue conducting monitoring work to establish robust datasets.

3.3 Data acquisition at Milford-on-Sea

3.3.1 The RF-PeBLE project: Risk-based Framework for Predicting Long-term Beach Evolution

The Risk-based Framework for Predicting Long-term Beach Evolution was a research project undertaken in the UK between October 2005 and February 2009 funded by the EPSRC through grant reference EP/C005392/1.The project aimed to develop an integrated risk based framework for the management of coastal systems to be demonstrated by application to a mixed, shingle and sand, beach (Simmonds et al., 2006). The proposed approach was under the frame of three methods integrated in the following activities: uncertainty and process modelling, system reliability framework development and field monitoring, data assimilation and analysis.

Thus, given the distinctive coastal elements present at Milford-on-Sea commented in the previous section, and the historical data available for this site, this mixed beach was elected as the site of the study.

The RF-PeBLE project data set consisted of, Figure 3.6:

- Sediment transport: Short-term impoundment experiment; represent the data set used in the work presented in this thesis and it is explained in detail in Section 3.4.

- Tide data: a tidal gauge deployed in Becton Bunny was recording tide data since March 2007 until the date that the Argus cameras were running. Also, an initial bathymetry survey has been done in an early stage of the project (5th February 2007) and the AWAC profiler (Acoustic Wave and Current) has been recording wave data from end of January to end of April 2007.

- Beach topographic surveys: since October 2006 to April 2008, monthly beach profile surveys.

- Argus Beach Monitoring System (ABMS) formed by five cameras mounted in a tower of 16m height facing the beach sited on the top of the cliff deployed in January 2007 until October 2011.

- Seasonal sediment samples across six elected profile lines representative for the alongshore extent considered.

- Bathymetry survey data.

- Wave data: WaveRider buoy offshore Christchurch Bay (CCO).

Figure 3.6 Aerial view from Becton Bunny to Milford on Sea. The layout of the RF-PeBLE project includes location of topographic survey lines, the Argus tower, tidal gauge at Becton Bunny, the offshore WaveRider buoy (CCO) the AWAC deployed.

The surveyed area covered a length of beach of approximately 3400m that extends from Milford-on-Sea to Becton Bunny. It is divided in 10 profile lines represented by the blue lines in the figure above, configuring irregular beach cells, they are denoted as 'MF' that stands for Milford-on-Sea.

3.3.2 Short term experimental data collection

The short term experimental data collection corresponds to the data collected during the time when a temporary structure was deployed at Milford-on-Sea over approximately two months, from 24th September 2007 to 30th November 2007. The experiment took place between the eastern side of Hordle Cliff and the western margin of Milford-on-Sea. The methodology applied is explained in Section 3.4.; the data collected during the experiment, complemented by the tide data recorded at the time by the tidal gauge deployed at Becton Bunny and the video images captured by the Argus cameras, form the data set used in this thesis. For the author's knowledge, it was the first time that this technique is applied in UK, indeed in a mixed beach (Reeve et al. 2004).

3.4 Methods of experimental data acquisition and analysis

3.4.1 Novel impoundment technique

3.4.1.1 Theoretical assumptions

According to the beach morphology methods explained in Section 2.2.5., the beach profile dataset surveyed are used to assess the plan shape evolution of the beach. It is assumed that from changes on the beach profile it is possible obtain the changes in the plan shape due to longshore drift effects looking at particular contour elevation, i.e. the shoreline at specific water levels.

This impoundment approach relies on the Principle of Mass Conservation applied through the Sediment Continuity Equation assuming that the shore normal temporary structure functions successfully as a barrier for the sediment in particular for shingle grain size, as it moves over or close to the beach surface and for this reason, a barrier to the longshore transport such as a groyne is an effective method (Brampton and Goldberg, 1991). The continuity equation refers the rate at which a part of the beach considered retreats or advances to the changes in the quantity of littoral drift in the longshore direction (Komar, 1976). This method will allow quantifying of the morphological changes. The reliability of the technique lies on the mass balance as accretion on the up drift side and erosion on the down drift side, as it is expected, but the volumes should be approximately the same always that the structure works with success (Wang et al., 1999). The trapping efficiency of the groyne can be evaluated by the variability of the cross-shore distribution of the longshore drift influenced by the tidal elevation which is considered for the groyne design (Brampton and Goldberg, 1991). Normally groynes do not reach the low water level and may even be porous (Rogers et al., 2010).

Thus, in this study it is assumed that the sediment balance between the deposition and loss of material in each cell is due to the longshore sediment transport under the influence of the variability of the wave conditions.

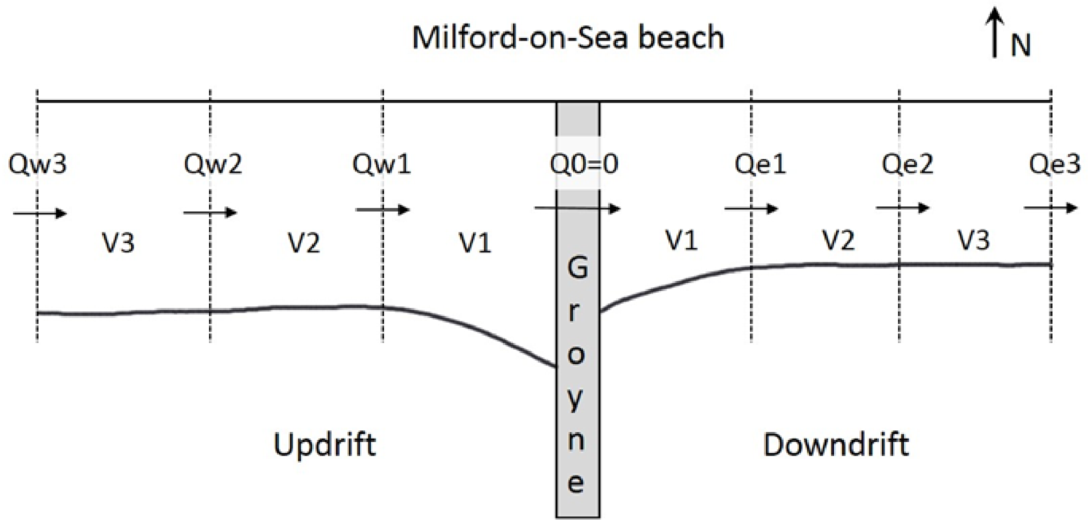


Figure 3.7 Schematic diagram of finite difference scheme for the volumetric survey data. The beach surveyed lines are represented by the black dashed lines; the notation 'w' and 'e' stands for West and East, 'Q' are the longshore sediment transport rates and 'V' volume.

Figure 3.7represents a simple schematic plan view of the beach profile data disposition to proceed with the volumetric transport rate calculations. It indicates the longshore sediment transport pattern transferred between the beach units (in-out) showed by the arrows pointed eastwards, based on the Principle of Mass Conservation. At the groyne, the transfer of mass is assumed to be 0, (*Q0=0*) what means that it is considered that the structure acts as a barrier and there is no sediment passing through.

In order to estimate the LST rates, the beach profile areas, A, are calculated in the first place, then the beach volumes between contiguous profile lines separated a distance *dx*, are calculated according to Eq. 3.1. Considering the alongshore direction on the x- axis, and the offshore distance in the y-axis, i is the location of the beach profile line on the alongshore direction, V the volume and t the time (date in a daily basis).

$$V_i(t) = \frac{A_{i-1}(t)+A_i(t)}{2} * dx \qquad \text{Eq. 3.1}$$

The areas and longshore distances are given in meters, thus the volumes are given in units of m^3.

The longshore sediment transport rates, Q, are calculated as the finite difference for the beach volumetric data per unit length of beach and per time, this is defined in general as shown by Eq. 3.2.

$$Q_i(t) - Q_{i-1}(t) = V_i(t+1) - V_i(t) \qquad \text{Eq.3.2}$$

The LST rate that is transfer to the beach cell 'i' at time 't' is represented by '$Q_i(t)$' and it may be refer as 'Q_{in}'. At the same time, the LST rate that is transfer from 'i' to the contiguous beach cell 'i-1' it is represented by '$Q_{i-1}(t)$' and may be denote as 'Q_{out}'.

Considering the volumetric survey data scheme for the groyne experiment represented in Figure 3.7, it is important to define the LST rate formulation at each side of the groyne, because the notation is reversed the Eq. 3.2. has to be re-written in the corresponding form for both cases. Then, assuming that the predominant sediment transport drift is eastwards, for the west side of the groyne the Eq. 3.2.canbe written in the form, Eq. 3.3.

$$Q_i(t) = V_i(t+1) - V_i(t) + Q_{i-1}(t) \qquad \text{Eq.3.3}$$

Following the same criteria, the formulation to apply at the East side of the groyne is given by Eq. 3.4.

$$Q_{i-1}(t) - Q_i(t) = V_i(t+1) - V_i(t) \qquad \text{Eq.3.4}$$

This Eq. 3.4.canbe written as follows, Eq. 3.5.

$$Q_i(t) = -\,(V_i(t+1) - V_i(t)) + Q_{i-1}(t) \qquad \text{Eq.3.5}$$

The estimations of the sediment fluxes using the Eq. 3.3 and 3.4 are in units of m^3 per unit of time, thus the flux has to be divided by 86400sec to transform to units of m^3/s (i.e. 1day, that it is the time scale of the surveys).

As an example, if $Q_i(t)$ is the sediment flux for the beach volume *i* at the time *t*, according to Figure 3.7and the Eqs. 3.2 – 3.4, for *i=1* and *t=1*, at the groyne location $Q_0(1) = 0$, and the LST rate at the West and East side will be represented by Eq. 3.6 and Eq. 3.7, respectively.

$$Q_1(1) = V_1(2) - V_1(1) \qquad \text{Eq.3.6}$$

$$Q_1(1) = -\,(V_1(2) - V_1(1)) \qquad \text{Eq.3.7}$$

These LST rates are used for calibrating the CERC equation (1984). Chapter 5 describes the method and the calculations made in order to estimate the longshore sediment transport coefficients of that formula.

3.4.1.2 Temporary groyne design

A temporary groyne was to be deployed for two months with the aim of impounding sediment moving along the coast. As mentioned before, given a predominant longshore transport direction positive from West to East, according to the dominant wave direction, and considering that there is no transfer of longshore sediments passing through the structure, it is expected that the groyne will trap the sediments on the updrift side, i.e. the west, and consequently, will cause erosion on the downdrift side.

The groyne was decided to be made of GeoTextile Bags, sometimes referred to as geobags. They were custom made to have 1m x 1m x 1m size and 2000Kg of capacity,

Figure 3.8. In the image it is observed that it is closed on the top and tied by a rope. The geobags were filled with native material picked up from the top of the beach and from different locations. Consequently, once the experiment finished, they were emptied spreading the beach material along the place where it had been taken.

Figure 3.8 Custom made GeoTextile Bag filled with native material from the beach.

The groyne was approximately 40m length (from survey location measurements) and it was designed and set out taking in account the mean water spring range, 2m, to ensure that the structure would cover whole the length of the swash zone and all the shingle upper beach. There are specific wave and tide data required for the structures design (USACE, 1992), however to design this experimental groyne mainly the water level variations and wave height have been considered. At any case the groyne did not get the bar and there was no influence of any other artificial structure over the stretch of beach considered. Figures A.2.1 and A.2.2 in Appendix A show a plan view diagram of the location of the temporary groyne at Hordle Cliff and a schematic cross section of the groyne design respectively.

Following the recommendations of the Shore Protection Manual (1984) and in order to achieve the function of the structure, interrupt LST, a straight stretch of beach of circa

300m had been set divided in two sections of circa 150m at each side of the groyne. This distance is more than the extent recommended by the SPM (1984) that relates a distance “on the order of two or three groyne length where this lengths is specified from the beach berm crest to the seaward end of the groyne.

3.4.1.3 Temporary groyne construction

The groyne construction commenced the 27th September 2007 starting for filling up the geobags. Overall, for the construction were necessary two diggers. As far as the geobags were filled using the native material from the top of the beach from a reasonable location far enough from the experimental area, Figure 3.9, and closed tied the top, Figure 3.10, they were aligned at the top of the beach, Figure 3.11.

Figure 3.9 Groyne construction: filling up and close tied the geobags.

Figure 3.10 Groyne construction: close tied the geobags as they were filled.

Figure 3.11 Groyne construction: geobags aligned along the top of the beach as they were filled up and close tied.

For the deployment of the temporary structure it was planned that the best location of the groyne would be along one beach profile survey line, denoted as GL, measured during the RF-PeBLE monthly surveys since January 2007, just below and aligned to the Argus tower. The structure was built in two differentiated parts and with a pyramidal section. The upper part of the groyne was formed by three heights corresponding to three geobags levels. The bottom one formed by three geobags wide; then, on the top of that, two geobags wide and finally, one geobag wide formed the top of that section. Continuing the groyne construction seawards, the second part corresponds to two height geobag levels; the lower part was form by two geobags wide and on the top of that, the second height formed by one geobag, Figure 3.12andFigure 3.13.

The lower part of the groyne was the first one of being deployed. It was during the low spring tide and it was necessary to dig 0.5m to place the geobags. All geobags, as far as they were deployed, they were tied up with a rope between them using the handles custom made at the top corners of the geobags.

Figure 3.12 Deployment of the lower section of the temporary groyne.

Figure 3.13 On the left hand side, geobags tied with a rope. Middle and right hand side pictures, upper section of the structure and an overall view of the groyne at the last stage of the construction.

After the deployment of the lower part of the groyne, the construction continued landwards with the upper section. Again, it was necessary to dig 0.5m to place the bottom level of the geobags. At the end of the construction, the beach area around the groyne was levelled to recover its natural shape whether possible to start the topographic surveys.

3.4.2 Topographic beach surveys

3.4.2.1 Survey grid layout

The experiment took place in western side of Milford-on-Sea close to the eastern margin of the beach of Hordle Cliff. A survey grid was designed in order to conduct the topographic measurements, this was set out to cover an extension of approximately 300m alongshore, Figure 3.14. The regular survey grid designed comprised 16 beach profile lines (red lines) approximately 10m spacing, at each side of the structure, including the two cross-shore sections surveyed along the groyne (black line). Points along the profile were set with a frequency of 1m, however, if any significant beach change or feature was observed in between it was also measured. The density of the profile lines are sufficient to provide an appropriate representative coverage of the beach.

Figure 3.14 Plan view of the experimental site at Milford-on-Sea (aerial image courtesy of the Plymouth Coastal Observatory, 2008).

While monthly beach profile surveys were carried out during one of the two spring tides that occur in a month, in order to cover the largest intertidal zone extent along the profile, the daily topographic surveys were simply conducted during the most convenient low tide considering weather conditions and day light duration.

The extension on the cross- shore axis was determined by tidal level, normally extended to the MLWS (Mean Low Water Spring) or the low water level of the day and beyond if possible, and the limited capability of the GPS- rover to work in wet conditions in the water, i.e. limit seawards up to the Trimble® Survey Controller (TSC2) that is attached to the survey pole did not reach the sea water as the electronic components cannot be submerged, Figure 3.15and Figure 3.16.

The beach profile lines were denoted as GW and GE referred to the 'groyne west' and 'groyne east' sides respectively, and numbered from 00 starting at the groyne to 15 for the last profile line at the western and eastern boundaries of the survey grid. Added to those profile lines, there is another one denoted as GL for ‘groyne line’, mentioned in Section 3.4.1.3., located between the groyne and GE01 and thought as the ideal location for the temporary structure. Following the same notation, the volumes corresponding to the beach cells defined by the profile lines are referred as VW and VE, ‘volume west’ and ‘volume east’ respectively, and numbered 01 to 15 starting at the beach cell delimited by the groyne and the first profile.

Figure 3.15 Surveyor measuring in the water, the yellow arrow indicates the survey controller attached to the survey pole.

Figure 3.16 Left and right: surveyor taking measurements in the water up to a point at which the TSC2 survey controller is not in risk of touch the water.

It is expected that the measured beach profiles change as a response to the wave conditions and water level variations. These changes can also be viewed as the plan shape evolution of the beach. Topographic analysis to identify the migration of specific contour levels allows the effect of the temporary groyne to be assessed and the mobility of the sediment in response to hydrodynamic forcings to be examined.

3.4.2.2 RTK-DGPS

The beach profiles were measured using a Differential Global Positioning System (DGPS) which provides a high level of accuracy in order to minimize the accumulated error for beach volume calculations, Figure 3.17. This system allowed measurement of beach profiles in the 3 Spatial Dimensions. Real Time Kinematic Global Positioning System (RTK- GPS) technique was used to conduct the beach profile surveys. The advantage of this technique is the capability for logging data at high speed and the accuracy for capturing the data, which is in the order of +/- 30mm on the vertical positions and +/-15mm on the horizontal.

There were three GPS receivers used: one GPS receiver was set as the base station and it was mounted on the Argus tower at 19.8m ODN high, to provide the corrections. The precision of the GPS base station was based on the RTCM standards (Radio Technical Commission for Maritime Services) as indicated by the competent operational procedure and uses standard OSTN02 transformations to provide Ordnance Survey coordinates. The other two GPS receivers were set as the rover units working simultaneously to measure the beach profile lines. The base station and the rover units were linked by radio signal using the SATEL external radio system.

Following common practices (Rogers et al., 2010), to ensure the same survey method the measurements were conducted by the same survey team. Three consecutive measurements of a known benchmark position located at the car park at circa 16.8 m ODN high, denoted as ‘control point 2’, were recorded by each surveyor prior starting to survey the beach profile grid to verify that the system was receiving the corrections adequately. The data was logging at 10Hz rate for better accuracy.

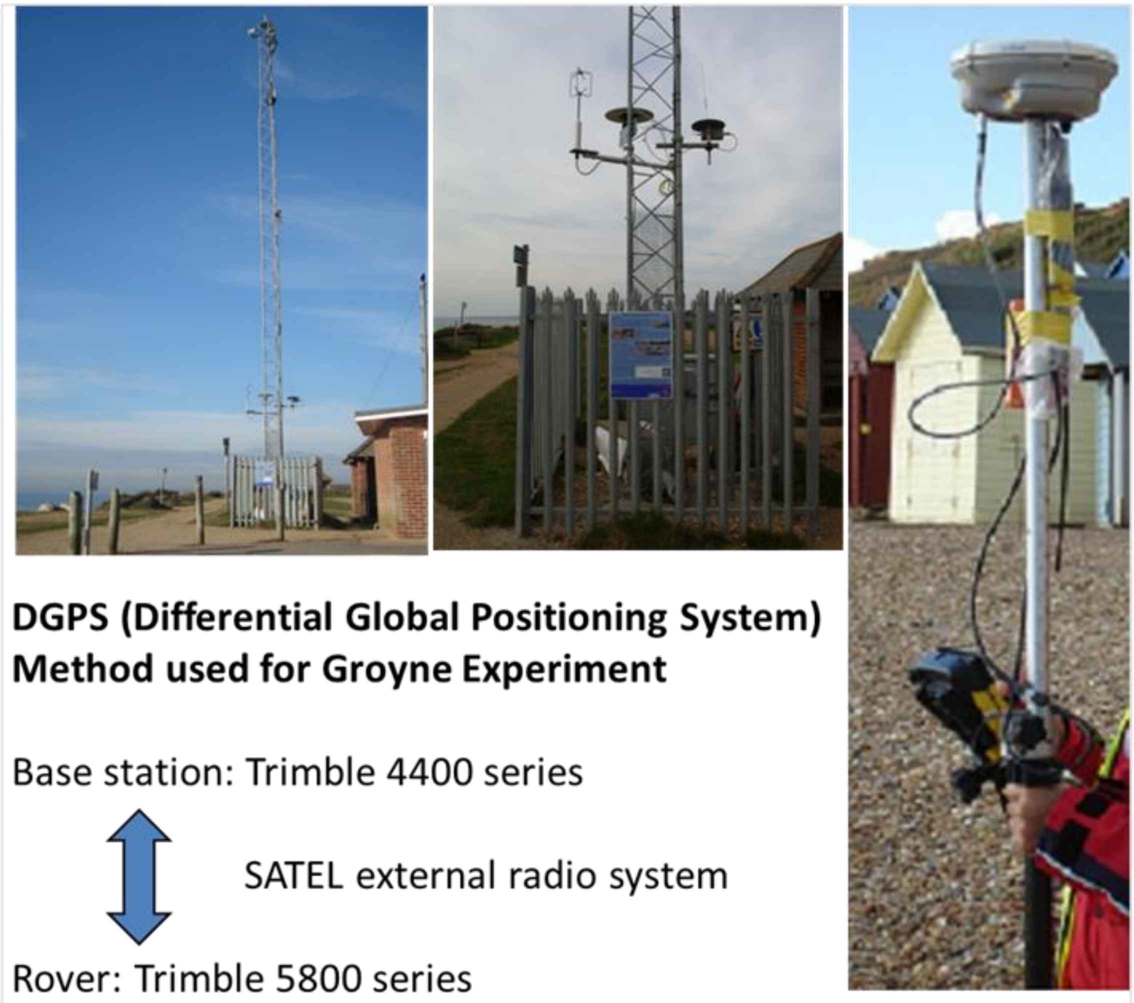


Figure 3.17 Left: Argus tower deployed at Hordle Cliff where the base station as part of the DGPS was allocated during the groyne experiment (centre). Right: DGPS Rover unit with a SATEL external radio attached and the DGPS controller unit. During the topographic surveys using the DGPS, the satellites corrections were transmitted from the base to the rover throughout the external radios.

3.4.2.3 Field feature observations record

Considering the capabilities of the Trimble TC12 controller, as part of the RTK- DGPS beach survey procedure the beach features and observations were recorded. The features code was interpreted according to a criteria previously established by the survey team for consistency of the method and to avoid possible variability due to human interpretations. Some examples of the beach features recorded were the water level marks, type of sediment according to its size or the boundary sections of the groyne (e.g.

High Water Level (HWL), Low Water Level (LWL), Mean Water Level (MWL), GS, SS, MX for gravel, sand or mixed sediments, CUSPS for points measured over cusps).

3.4.2.3.1 Cross- shore shingle- sand interface criteria

It is assumed that at Milford-on-Sea the coarse size sediments act as the principal mechanism for the coastal defence against the action of waves and tides. According to this, it is thought that the shingle fraction moves up and down along the beach face over a terrace of sand influenced by the hydrodynamic forces. Thus, there is a natural boundary established by the change in sediment grain size along the profile, i.e. the limit between the shingle fraction and the sand. This boundary is referred to as the interface shingle-sand.

The position of this interface can be defined and tracked using the sediment feature codes recorded.

3.4.2.3.2 Correlation method between the beach elevation and the cross-shore position of the interface

Considering that the beach face is a function of the grain size, the degree of sediment sorting, the effect of the wave energy and the tidal cycle and stage (Komar 1976), and once that the interface position is defined, it is thought that its location should vary along the profile according to the variation of the water level. This assumption is intended to be demonstrated by applying a bivariate analysis to the data, relating the cross-shore position of the interface to its elevation.

Thus, by fitting a linear model to the data it is expected to provide evidence of the expected dependence between both variables and relate those to the tide cycle. The measure of the strength and the tendency of the linear relationship is described by the estimation of the Pearson's correlation coefficient, known as '*r*', that is one of the most common coefficient used to compare quantitatively the two variables (Dyke, 2007). The coefficient is defined as the covariance of the two variables divided by the product of their standard deviation and it varies between 1 for perfect agreement, 0 for not relationship and -1 for total negative correlation or perfect disagreement.

In this case, as the elevation is reduce and reaches values below 0 with the offshore distance, it is expected to obtain a negative linear relationship that should give a negative correlation coefficient.

3.4.3 Wave and tide data

Simultaneous measurements of wave and tide data were measured at the same period as the beach profile surveys. Waves were recorded using an AWAC (Acoustic Wave And Current) current profiler that gives wave height and directional spectrum, Figure 3.18(left). The instrument was deployed at 50º 43. 209 N Latitude, 001º 36. 970W Longitude (Figure 3.6) the 28/08/2007 and it was set up to start recording the 04/09/2007 at 9am for 100 days. The wave sample rate was configured for logging at 2Hz and the wave interval 3600s, then there were wave data records every hour. The AWAC wave data is recorded respect to the geographic North. As the beach normal is approximately 108º from North, the wave direction α measured by the AWAC must be adjusted with respect to this in order to get the correct alongshore wave power.

The tides have been measured with a tidal gauge RBR TWR-2050 Series that measures tidal elevation and wave height and period, Figure 3.18(centre). Two tidal gauges have been deployed in the site of study at two different locations. One, was deployed only during the groyne experiment, attached to a pole at the end of the experimental structure, Figure 3.19, and was recording data from 29th September to 16th November 2007 at 10 min intervals; it was recovered at the same time as the groyne was pulled out. However, the other tidal gauge was deployed previously, in March 2007, at the seaward end of an old sewage out fall pipe in Becton Bunny, at the western side of Christchurch Bay recording data each 15min.

Figure 3.18 Left hand side: shows the AWAC; centre: RBR TWR- 2050 Series tidal gauge deployed at the seawards end of the temporary structure. Right: the Zephyr vessel, a Cheetah Marine 6.9m Catamaran (CCO) conducting the bathymetric survey the 28th August 2007 during the groyne construction.

The tide data at Becton at the time of the experiment corresponded with the tidal gauge deployments 3 and 4. The wave sampling was enabled at a rate of 2Hz, the length 2048 and the averaging was set to 300s. The sampling period was 15min. The raw data from the logger memory is exported using the instrument software. The tide file consists of water temperature and pressure at the sensor depth. Another pressure sensor (*Patm*) deployed in the cabinet of Argus tower was recording the atmospheric pressure. This sensor logged data every 30minutes. These data were re-sampled to the same frequency as the tide sensor to take out the effects of atmospheric pressure on the pressure at the tide sensor (*Ptide*). Thus the actual pressure (*P*) at the tide sensor is calculated as the difference between the *Ptide* and *Patm*. The corrected pressure *P* is converted to corresponding water depth by using the relation '1 decibar pressure equals 1 metre of water'. Once the water depth is obtained the last correction is to refer the data to National Grid Transformations OSTN02 Survey datum that is -1.807m for the tidal gauge at Becton Bunny, whereas the corrections applied to the data recorded by the tidal gauge at the end of the groyne were -1.605m and -1.509 for the instrument deployment 1 and 2, respectively. Then, the tide elevation in mODN is represented by η, Eq. 3.8.

$$\eta = P + correction \qquad \text{Eq. 3.8.}$$

Figure 3.19 Milford-on-Sea view from the seawards end of the temporary groyne; the tide gauge is attached to the pole observed at the right of the image, for security is tied up to the groyne. The Argus cameras are installed in the higher tower at the top of the cliff that is aligned with the groyne.

The contemporary measurements of wave data are intended to be used to estimate the longshore wave power according to Eq. 2.6 and fit into the CERC equation, Eq. 2.1. As the AWAC was deployed at intermediate- shallow waters it is necessary to conduct a wave transformation analysis to know the wave conditions at breaking. Thus, continuing with the empirical approach adopted in this study, the Small Amplitude Wave Theory is considered to conduct the wave transformation in shallow waters to the wave data. This analysis is presented in Chapter 4.

Tide data are used to investigate the effect of the water elevation on the profile changes. Considering the relevance given to the effects of tides to the beach profile variability for mixed, shingle and sand, and gravel beaches by different authors, as seen in Chapter 1 and Chapter 2, the tide data is integrated in the analysis relating the elevation of the tide to the interface position. Also, the tidal data is integrated into the one line model applied that is presented in Chapter 6.

3.4.4 Bathymetric surveys

Normally nearshore bathymetric survey data are used to overlap the beach profile to an extent out to a depth where the wave induced sediment transport may be negligible (Rogers et al., 2010), but given the particular technology that is required to conduct the hydrographic surveys, those are less frequent than beach profile surveys. In this study three bathymetric surveys were conducted by the Coastal Channel Observatory (CCO): bathymetric survey in February 2007 planned to cover the alongshore extension investigated by the RF-PeBLE project; the bathymetric survey of August 2007, to establish the pre- experiment bathymetric conditions and this comprises the bathymetric data to be used in the one line numerical model (in Chapter 6); and the bathymetric survey post-experiment at the end of November 2007. The hydrographic surveys were conducted using a Raytheon single beam echo-sounder (model DE719E 200 Khz standard transducer- 8º beam), the draft of the survey vessel was 0.4m and it was sampling at high frequency pulse of 200Khz. The quality information provided noted that the estimated positional accuracy of the dataset was +/- 1m and the estimated depth accuracy +/- 0.3m, typically +/- 0.1m; no heave compensator was used, the tide correction were determined from on board RTK- GPS and the surveys were conducted in calm conditions.

In order to apply the linear wave theory it is expected that the nearshore bathymetry data shows a seabed with a gentle slope with approximately parallel contours.

3.4.5 Beach sediment sampling for grain size analysis

To determine the sediment sizes and sediment size distribution at Milford-on-Sea, four data sets of surface sediment sampling were collected along six selected beach profiles. In order to collect representative samples of the survey grid, the profile lines chosen were located, for each side of the temporary structure, near the groyne, approximately at a middle distance longshore and close to the more western and eastern limits. The sediment samples were collected from the same location along the profile for each data set and during the spring tide. The frequency of the sampling positions was set between 5-10m spacing from the top of the beach up to a point located at the low water spring level and beyond.

The surface sediment samples were collected using a spade along the elected transects from the top of the beach. The range of grain sizes of a sample, was determined by sieving the sediments, a technique that is the most common method for the analysis. The sieves consist in different pans of a standard mesh, in this case the British Standard mesh, made by a wire screen, arranged in a stack. Once the sample is placed in the top of the sieve, these are shaken to make the sediments fall through the stack. As result, the sediments are divided in different size fractions that have been trapped by the sieves of distinct mesh sizes. That allows determine the weight of sediment caught by each band and consequently also the percentage of the total weight of the sample passing through the sieve (Dean and Dalrymple, 2004). The standard sieves used for the sediment samples collected at Milford-on-Sea varied between 0.063mm for the finer fraction to 50mm for the coarser sediments.

The sieving analysis measures the length of the intermediate axis of a grain size, denote by convention the b-axis (Masselink et al., 2011). Then, the grains are normally classified according to their b-axis length, the Wentworth classification scale is one of the more widespread schemes used (Dean and Dalrymple, 2004) which classifies the grains size using millimetres-scale and phi-scale (ϕ- scale) related by the following conversion, Eq. 3.9.

$$\phi = -log_2 D \qquad \text{Eq. 3.8}$$

Where D is the grain diameter given in mm. Large negative phi-values represent coarser grain sizes whereas positive phi-values indicate finer grain sizes. The reverse conversion is given by Eq. 3.10.

$$D = 2^{-\phi} \qquad \text{Eq.3.9}$$

Within the different ways to represent the sediment size, there is one that shows an experimental representation of the size distribution of sediment sizes of the sample, where the y-axis represents the sample percentage by weight between the sieve sizes, and the x-axis shows the size. However, one of the most common ways of representing the sediment size is the cumulative size distribution. That statistical plot shows "the value at a particular diameter that represents the total sample percentage by weight that is coarser than that diameter" (Dean and Dalrymple, 2004).

3.5 Discussion and conclusion

This chapter presents the assumptions made and the methodology applied in this study to investigate the longshore sediment transport rates at selected mixed beach of Milford-on-Sea. From observations it is thought that the beach presents a bimodal sediment size distribution, being the shingle content significantly higher than the percentage of finer sediments, and the main natural mechanism to protect the coast against adverse hydrodynamic conditions. It is expected that the beach responds as a gravel beach under the hydrodynamic forces and the shingle fraction moves onshore- offshore along the profile over a terrace of sand.

Thus, it is intended that the study focuses on the shingle fraction of the beach profiles surveys, defining those by the boundary established at the transition of shingle to sand sediments. Then, the longshore sediment transport rates are estimated respect to the shingle and the tidal elevation.

It is noted the importance of the profile extent that is considered in order to cover the range of data that is representative of the profile change and no data are missed, this is, the inclusion of all the range of action where the longshore sediment transport is taking place and prone to be measured throughout this method. For this reason, a conventional analysis of the beach profiles surveyed should be conducted, e.g. considering the whole range of data measured and calculate the areas respect to a specific elevation, i.e. the tidal levels. The comparison of both analyse should provide information about the importance of the profile extent for the effects on the shoreline change as the analysis of the shingle fraction scopes out the longshore bar measurements when they were surveyed and they may have notable effects on the shoreline position (Farris and List, 2007).

Chapter 4

4 Longshore sediment transport on Milford-on-Sea

4.1 Introduction

This chapter presents the analysis of the field data and results. Preliminary results are presented in Section 4.2.1 from observations in situ and the practical knowledge of the site acquired while conducting the surveys. Sections 4.2.2 to 4.2.8 present that analysis of the data collected according to the methodology explained in Chapter 3. At the end of September 2007 the temporary groyne structure was deployed; then, since the 1st October until 24th November 2007 daily topographic surveys were performed on one low tide of each day. At the end of the experiment time, a data set of 33 profile lines, i.e. 16 profile lines at each side of the groyne were measured for 56 days, being a total of 1848 profile lines surveyed, complemented by the contemporary wave and tide data also recorded at the site.

Section 4.3 summarised the discussions and conclusions raised from the analysis. Data processing suggest a revision of the survey procedures; the results put forward for consideration the method for analysing the shingle sediment fraction as an approach for further development regarding the different assumptions adopted, from the criteria for defining the interface to the determination of the 'active' and 'no active' transport.

4.2 Data analysis and results

4.2.1 Beach features

Milford-on-Sea comprises a mixture of sediments dominated by coarse size sediment acting as the main mechanism to protect the cliff. The beach shows gravel size above high water level, mixed sediments in inter-tidal beach, sand size below low water line, a longshore bar exposed during springs tides and cusps formed under low energy and shore normal wave incidence, Figure 4.1. According to these characteristics, Milford-on-Sea may further be identified as a composite mixed beach (Mason et al., 1997) that is analogous to the "composite gravel beach" according to the morphodynamic model proposed by Jennings et al. (2002) based on the study of 42 gravel beaches of the South Island, New Zealand. In general Milford-on-Sea presents two berms, although there were dates when there could be observed up to three berms.

Figure 4.2 shows the surface of the sandy terrace that was assumed as the interface shingle- sand boundary to determine the beach profiles of shingle fraction. It is observed a distinct transition between the coarse sediments over a layer of composite mixed sand and shingle sediments.

Figure 4.1 Characteristics of field site, Milford-on-Sea. On the left-hand panel, sandy bar exposed during spring tide; centre, mixed (shingle and sand) sediments; right-hand panel, coarse grain size on the foreshore slope over a low terrace of sand looking eastwards.

Figure 4.2 Interface shingle- sand pointed by the yellow arrows showed when digging about 0.5m deep during the groyne construction.

The break-point step defined by Kirk (1980) was identified while conducting the surveys, and in agreement with his observations, it is characterised by coarse sediments, then just beyond this point, the lower terrace of sand extends seawards. It was observed that waves break at the longshore bar located closest to the shore during the low tide, to break again at the shoreline. This was observed significantly as the tide rises when waves tend to develop again after breaking on the longshore bar and approach the shoreline, where they break again.

Beach cusps were present the 80% of the time of the experiment; the other 20% of the time they were not formed or they were not clearly defined. Therefore, it was assumed that cusps are common beach features at Milford-on-Sea and they were integrated in the analysis within the beach topographic surveys. Sediment sorting over the cusps was observed to be in agreement with Nolans et al. (1999) findings. They presented coarser sediments on the cusp horns and finer sediments in the bay, Figure 4.3.

Figure 4.3 Cusps formed at Milford-on-Sea.

4.2.2 Groyne performance

In agreement with the assumptions made in Section 3.4.1, during the experiment period, there were observed beach morphological changes due to the presence of the temporary groyne related to the wave conditions. Given a predominant longshore sediment transport movement being positive on the eastwards direction, it was possible to observe updrift accumulation and downdrift erosion at the groyne location, Figure 4.4. In the same way, when the net sediment transport direction reversed, this trend was observed in the contrary way. Thus, these changes in the beach morphology and the observations of the wave conditions, suggested that the profile responded to the hydrodynamic forcing conditions. It is expected that the analysis of the beach profile surveys, wave and tide data demonstrate the preliminary observations presented in this section. The updrift accumulation is evidence of the littoral drift and it can be quantified with the estimation of volume changes related to the longshore wave power.

Figure 4.4 Downdrift at the left hand side and updrift at the right hand side of the structure, image taken the 8th November 2007.

In general, field observations suggested, qualitatively, that there is a link between beach profile changes and sea weather conditions; the formation of cusps related to normal wave incidence in calm conditions, accretion and shoreline advance on the updrift side and retreat of the shoreline on the downdrift, in occasions occurring with profile steepening just at the adjacent beach cell to the groyne at the East side; and the movement of the breaking line influenced by the tides.

The groyne was pulled out the 26th November 2007 coinciding with the spring tide. It took two days to remove the whole structure and two days more to return and spread the beach material on the beach from the geobags. A storm occurred onthe 28th November that contributed to redistribution and washing of the sediments that had been used for the structure and helped to accelerate recovery of the beach slope to its natural shape at the location where the groyne had been installed.

On the 8th October there were calm wave conditions and there were cusps formed at the beach, Figure 4.5 (left). From the 10th until the 15th, it was observed that the wave height increased slightly, the wave angle was predominantly from the East direction which caused a net sediment transport from east to west. It is expected that the estimated longshore wave power is negative for that period indicating the westwards direction of the littoral drift. This can be observed in Figure 4.5 (centre), on the 14th October there was sediment accumulation on the east side of the groyne and consequently, erosion at the west side. It is particularly noted by the shingle fraction that is retreated at the west side respect to the east of the groyne. On the 16th October, the storm conditions implied an increase in the wave height and the wave angle was consistent from the west, Figure 4.5(right). According to these observations it is expected that a positive longshore wave power traduced into a significant energetic event. Those conditions caused the erosion at the east side while accretion was expected at the west side of the structure.

Figure 4.5 Left to right: images of the temporary structure corresponding to the 8th, 14th and 16th October 2007.

The second significant energetic event observed took place during the 27th-28th October, Figure 4.6(left). A relevant increase of wave height was observed in a consistent southwest wave direction. This storm caused certain damage to the tied rope of the geobags of the upper part of the low section, Figure 4.6 (centre and right). The affected section of the groyne was repaired the following days.

The beginning November, from about the 4th, was characterised by calm conditions, and it was observed that the waves approached from the east direction. Thus, some sediment accumulation was noted at the east side of the groyne indicating a sediment transport drift to the west. However, the opposite was observed between the 5th and 11th of November when updrift accretion occurred at the west side of the groyne, which suggested that the littoral drift reversed towards the east, Figure 4.7. During those days, an increase in the wave height was also observed as well as a westerly wave direction, conditions likely to cause a significant longshore wave power.

Figure 4.6 Left to right: images of the temporary structure corresponding to the 28th, 29th and 31st October 2007 respectively.

Figure 4.7 Left to right: images of the temporary structure corresponding to the 5th, 7th and 8th November 2007.

It is thought that the analysis of the field measurements, i.e. the estimation of the longshore transport rates and shoreline changes from the beach topographic surveys, as well as the characterization of the wave conditions during the experiment and the estimation of the alongshore wave power, evidence the preliminary observations and support the assumption made that the changes in the beach profiles are due to longshore sediment transport processes.

4.2.3 Bathymetry mapping

The information of the depth contours in the nearshore zone was provided by the bathymetric surveys, important due to its effects to tidal currents and wave transformation subject to wave refraction, diffraction and shoaling processes. From the bathymetry it is possible to identify the development of seabed features, e.g. the longshore sand bar exposed during the spring tides at the study site, and a second submerged bar offshore, as well as to investigate the nature of the isobaths. The bathymetric data normally complement the beach topographic measurements which may be extended offshore, and this information is also important to model the wave propagation from deep waters to shallow waters.

Then, the depth contours in the nearshore zone at Milford-on-Sea are assessed qualitatively from the bathymetric measurements where the x-axis represents the longshore extent (Easting, m) and the y-axis the distance offshore from the coastline (Northing). In order to represent the bathymetry, the contour plot applied linear interpolation to the raw data to define a regular grid. The linear interpolation was considered an appropriate method given the high frequency sampling of the data measurements.

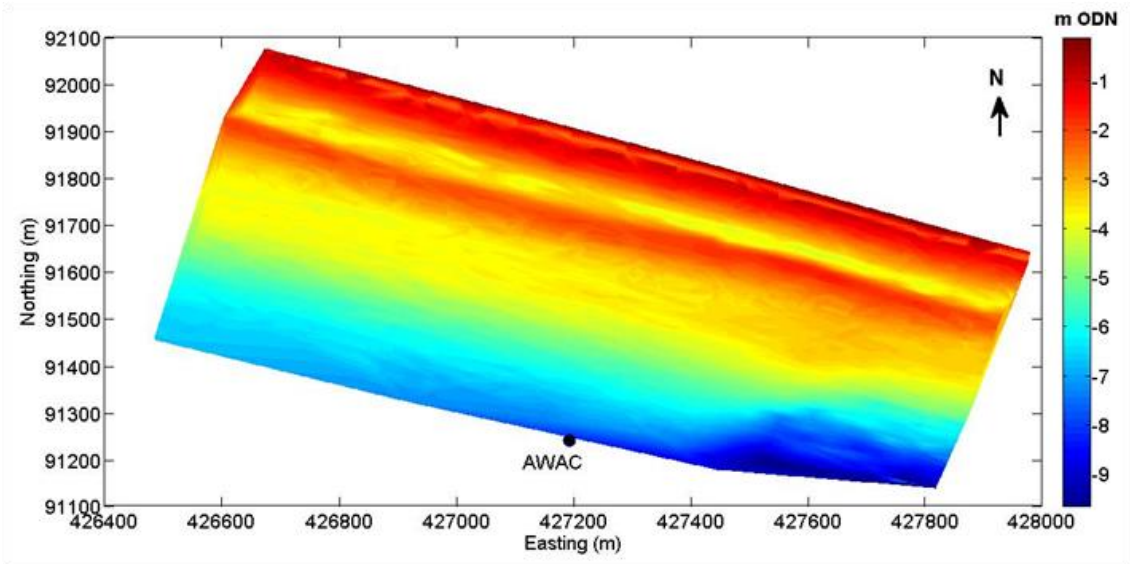


Figure 4.8 RF-PeBLE project bathymetry survey conducted in February 2007.

Figure 4.8 represents the bathymetric survey conducted in February 2007 that covers the longshore extent considered in the RF-PeBLE project. Despite that the AWAC instrument was not deployed at the time of this survey, it is included in the figure to have an approximation of the dimension of the area surveyed in comparison with the pre and post experiment hydrographic surveys, and to assess possible depth changes at that location. The elevation contours are indicated by the colour scale changing from red to blue with depth. Thus, it is possible to distinguish the shoreline contour in dark red at the north boundary of the grid, and the alongshore sand bar located near to the shore. From there and following the contours seawards, after the distinct -4m ODN contour line, the presence of a second longshore bar is noted with approximately -2m ODN depth. The data show that the seabed relief presents parallel contour lines.

The bathymetry surveys pre and post experiment were conducted at the end of August and at the end of November 2007, Figure 4.9and Figure 4.10,respectively, and they encompassed a longshore extent long enough to cover the length of the survey grid designed for the groyne experiment. The position of the AWAC shows that the instrument was deployed at approximately -7m ODN depth. It is observed that the isobaths in both, the pre and post survey data, present a similar pattern characterised by parallel depth contours and the presence of two longshore bars. Although, it is noted that the nearshore longshore bar is more developed and presents continuity along the contour level in the pre-experiment survey than in the post-experiment data, which show disruptions in the elevation along that contour line.

The bathymetry data conducted prior the experiment was used in the one line model simulations to add the effects of the tides in the shoreline changes. The approximately parallel contours of the seabed in shallow waters suggested that the application of the Snell's Law method to transform the offshore wave conditions up to the breaker line is an appropriate option. This consideration is consistent with the method applied by Brampton (1993) for modelling the wave propagation at Christchurch Bay from the -5m ODN contour to the breaker line.

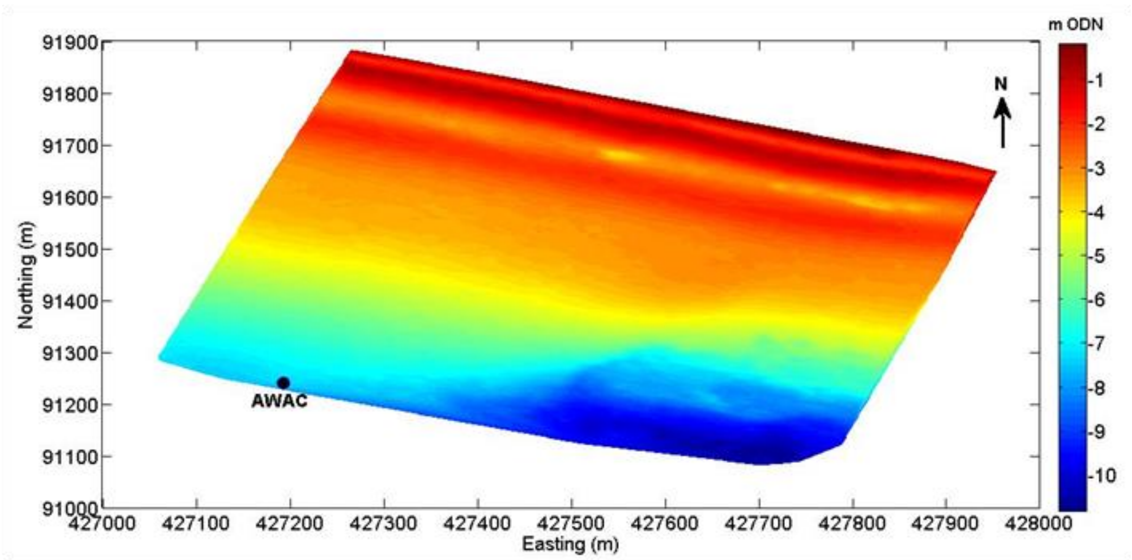


Figure 4.9 Bathymetry contour map surveyed the 28- 08- 2007, corresponding to the 'pre- experiment' baseline conditions.

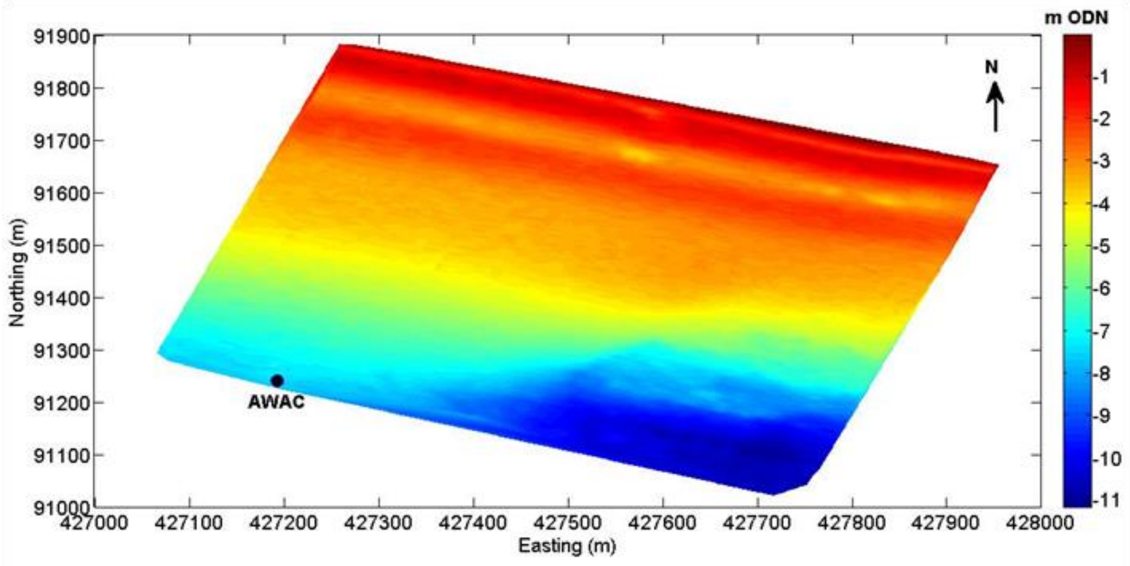


Figure 4.10 Bathymetry contour map surveyed the 26-11-2007, corresponding to the 'post-experiment' survey.

4.2.4 Tide data

The tide information was provided by two tidal gauges deployed at the site of the study as was described in Chapter 3. The tidal gauge at Becton Bunny recorded data at 15minute intervals and there is no data gap for the period of the experiment. The tidal gauge deployed at the end of the temporary structure was deployed at the same time as the temporary structure and it was recovered also the same day that the groyne was pulled out. This tidal gauge was set up for recording data at 10minute intervals however there is a gap of 8 days in the dataset because the last data recorded is for the 16th November 2007. For this reason, it was thought to use the complete tide dataset measured at Becton Bunny rather than the tide data measured at the groyne location. To demonstrate that this consideration is appropriate, i.e. to ensure that the two tide elevation data are in phase, a cross- correlation analysis was conducted between the two tide datasets.

The cross-correlation method is the correlation between a pair of time series variables, e.g. the tide elevation, where the values are paired by occurring at the same time. The correlation is in good agreement when the maximum strength of the signal is at lag 0.

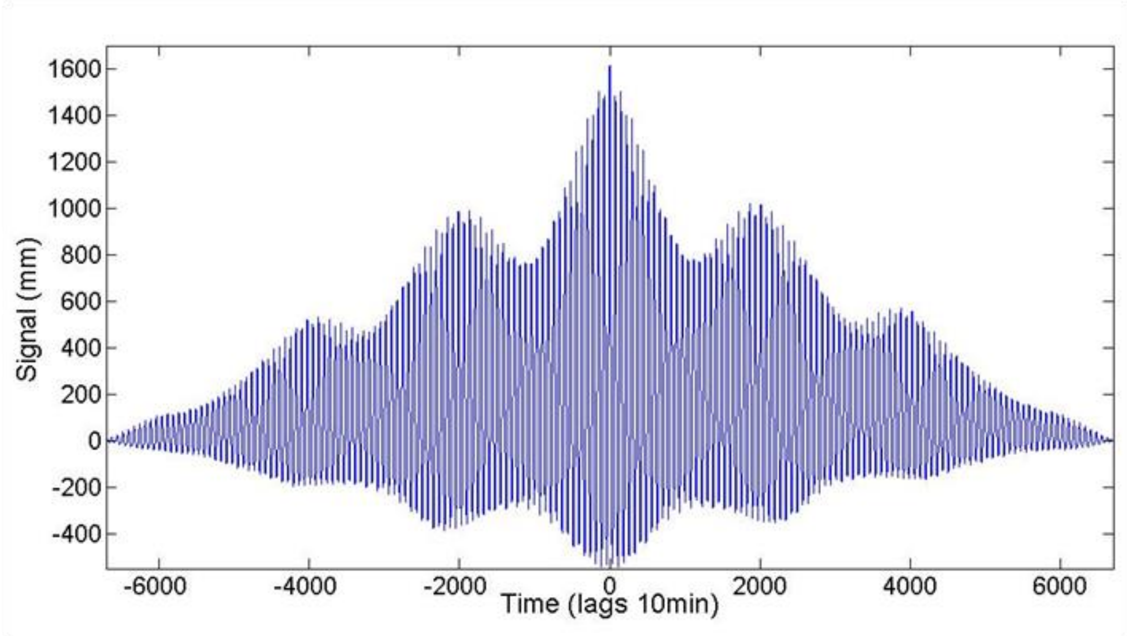


Figure 4.11 Cross- correlation between the tide data measured at Becton Bunny and at the temporary groyne.

In order to correlate the tides, the tidal data at Becton was re-sampled to a frequency of 10minutes, the same frequency as the tide measured at the groyne. Then, Figure 4.11shows the correlation between the two tide datasets, the x- axis represents the time in lags of 10min that is the sampling frequency, and the y- axis represents the signal (in mm) which is the measure of the strength of the cross-correlation. From the results it is observed that the maximum strength is given at lag 0, which it means that the two tide dataset are in phase. Consequently, it is accepted that the tide measured at Becton Bunny, Figure 4.12, is appropriate for the analysis of the beach profiles, presented in Section 4.2.7, and also to be used in the one line model simulations presented in Chapter 6.

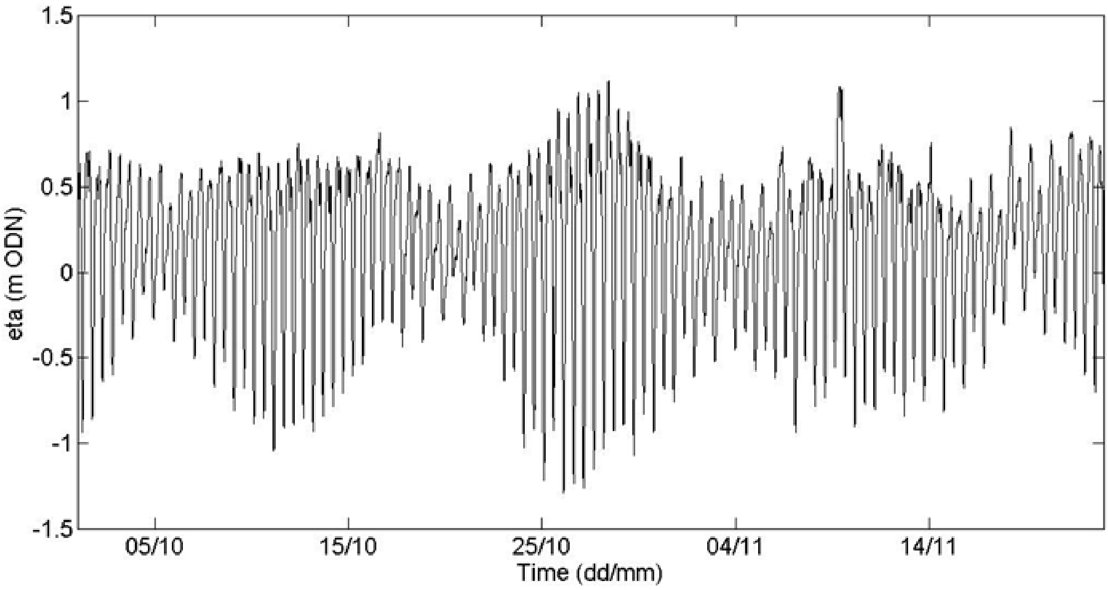


Figure 4.12 Tide data measured at Becton Bunny.

Figure 4.12represents the amplitude of the tide (eta, m ODN) recorded by the tidal gauge at Becton Bunny during the experiment period. It is observed that there were five spring tides corresponding with the time intervals of larger difference between the high and the low tide, and four neap tidal cycles corresponding with the time intervals of smaller difference between the high and the low tide. There are noted two large surges in the measurements, those are changes in the water level due to meteorological forcing (Rogers et al., 2010). One was measured the 9th November during the spring tide, it may be due to the combination of the wind forcing or significant changes in the atmospheric pressure. The second one the 18th November, date characterised by one of the storm events occurred during the experiment, therefore it is suggested that this positive surge was due to the combination of high wave height, flood tide and low atmospheric pressure.

4.2.5 Sediment distribution

In order to provide evidence of the bimodal sediment distribution at Milford-On-Sea, and to estimate the percentage by weight of the sediment size, four data sets of sediment samples were collected: one at the beginning of the experiment, two during the experiment and the last one, the first spring tide after the experiment period in

December 2007. The sampling was conducted according to the methodology presented in Chapter 3, however, it was not always possible to collect samples from all the locations, particularly from the points located near the low water level due to adverse wave climate conditions and time limitations. In agreement with Kirk (1980), the sieve analysis confirmed the variability of the grain size observed across the profile. As was expected, the finer fraction is present in the locations sampled within the intertidal range, particularly beyond the mean water level, nevertheless, the coarser fraction is always predominant over the finer fraction.

The sieving analysis of the surface sediment samples collected revealed that the sediment distribution across the beach profiles at Milford-On-Sea is characterised by presenting pure coarse grain size in the range defined between the top of the beach and the high water mark; and mixed shingle and sand grain size at the locations influence by the mean water level and beyond, e.g. sediment samples A1 to C1 and D1 to F1 respectively in Figure 4.13. It shows the location of the sediment samples A to F along the cross- shore profile corresponding to the data set collected at the beginning of the experiment (indicated by ‘1’).

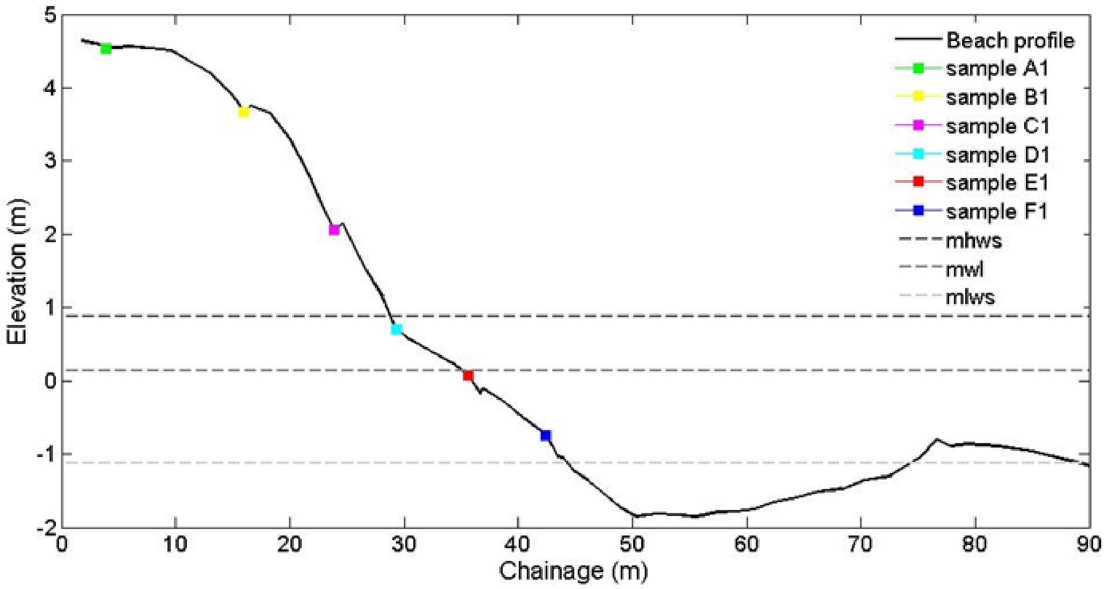


Figure 4.13 Location of the sediment samples along the beach profile. This example corresponds to the sampling conducted over the profile line GW01 the 30th September 2007.

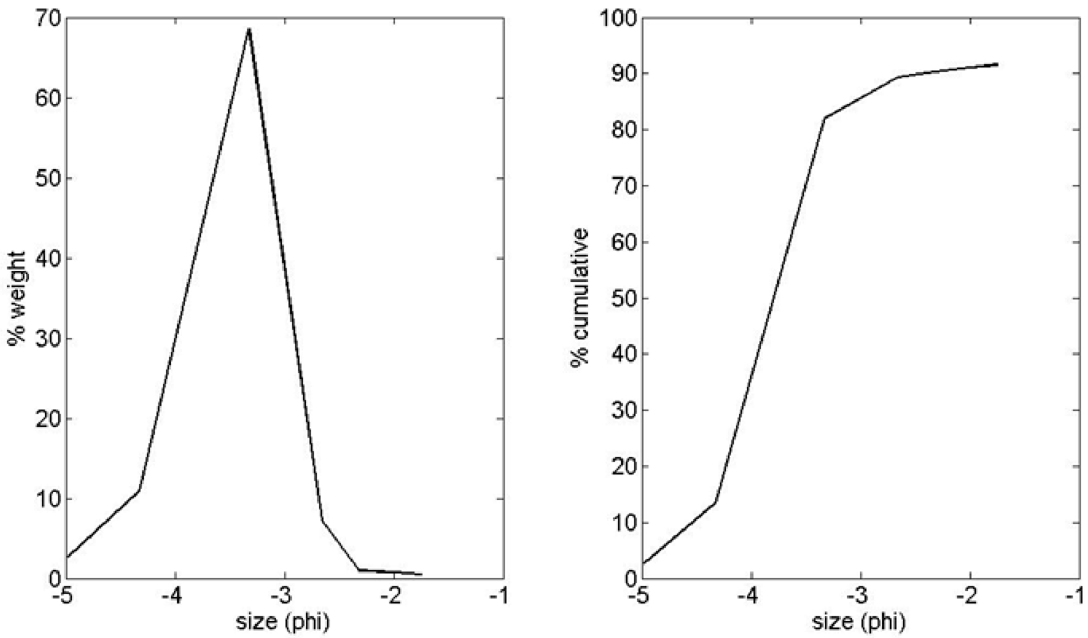


Figure 4.14 Example of sediment size distribution by weight (left) and percentage of cumulative frequency (right) in phi units for the sample A1 collected at the top of the beach profile line GW01.

As expected, the pure grain size samples present unimodal size distribution giving a skewness zero, which indicates that it is perfectly symmetric, typical of a normal distribution. Figure 4.14(left) represents an example of this size coarse size distribution. The estimation of the standard deviation of the grain size distribution indicated that the sediments are well sorted typical of reworked sediments (Masselink, 2011). Because the sorting refers the range of sizes in a sample, this result is expected for a sample of pure coarse size. The percentage of the cumulative frequency, Figure 4.14 (right) represents the cumulative frequency distribution of the grain size within the sample.

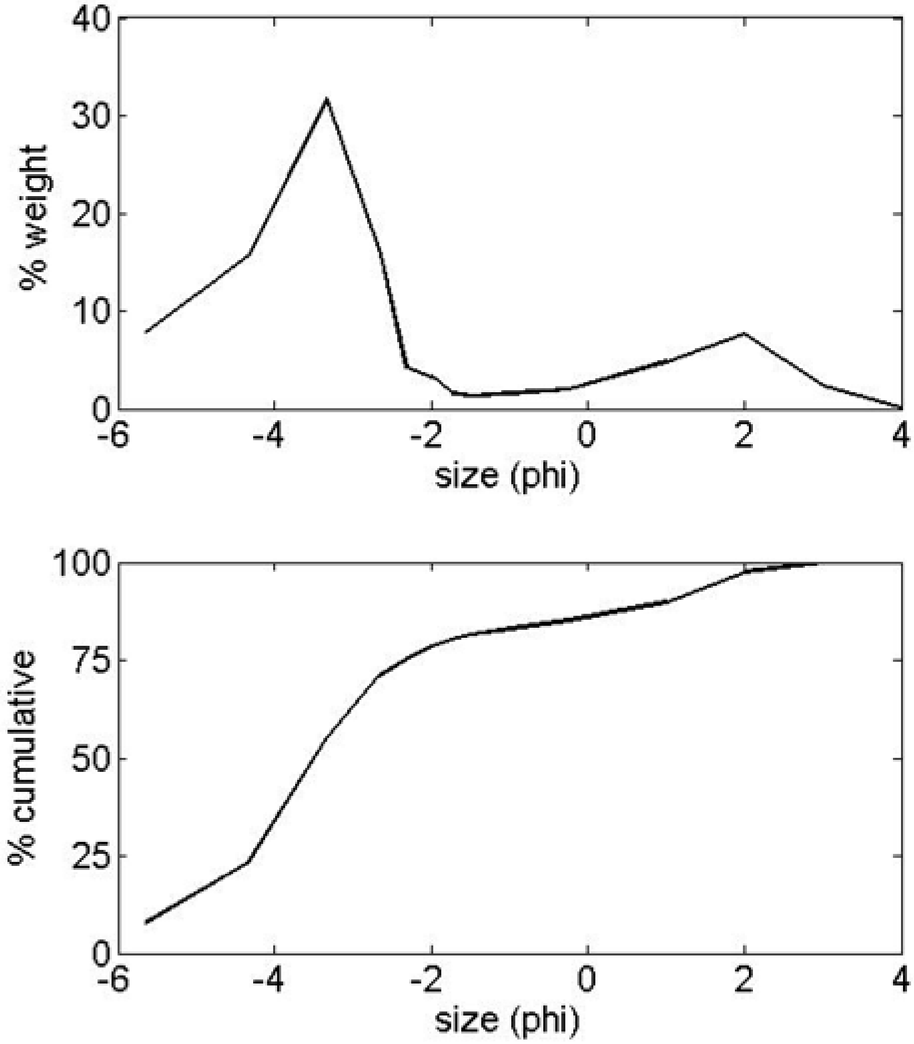


Figure 4.15 Example of sediment size distribution by weight (top) and percentage of cumulative frequency (bottom) in phi units for the sample A1 collected at a location near the MWL at profile line denoted GW01.

The mixed shingle and sand sediment samples gave a bimodal distribution indicated by the two peaks in the weight frequency, Figure 4.15(top). The skewness resulted was zero what indicates that the distribution is nearly symmetrical and the standard deviation showed a poorly sorted distribution. The latest is also indicated by the percentage of cumulative frequency distribution of the grain size, Figure 4.15 (bottom), it is typical of bimodal sediment size distributions.

According to the sieve analysis, the range of sizes measured at Milford-on-Sea in terms of the mean grain size varied between 33mm the coarsest and 0.3mm the finer. From the sediment analysis conducted it can be concluded that the mean sediment size for Milford-On-Sea is D50 = 10mm as representative of the study site considering the distribution by weight frequency. The sediment size is used in the one line simulations.

4.2.6 Wave data

The wave parameters measured by the Nortek Acoustic Wave and Current profiler, AWAC consisted in hourly: significant wave height *Hs*; mean wave period, *T(s)*, which is based on the moments of the energy spectrum; *Tp* peak wave period (*s*) that is the period associated with the frequency that has the most energy; mean wave direction (degrees) measured respect to the geographic North; peak wave direction (degrees) associated to the peak frequency; and peak wave spreading (degrees) what indicates the variance with the directional observations.

The analysis of the wave parameters allowed to determine the wave climate conditions in the nearshore zone during the experiment period. The data indicate that mean Hs= 0.44m within a range values between $0.01\text{m} \leq Hs \leq 2.12\text{m}$, Figure 4.16; mean T= 2.7s between $1.07\text{s} \leq T \leq 6.24\text{s}$, and mean Tp= 6.73s between $1.23\text{s} \leq Tp \leq 17.28\text{s}$, Figure 4.17.

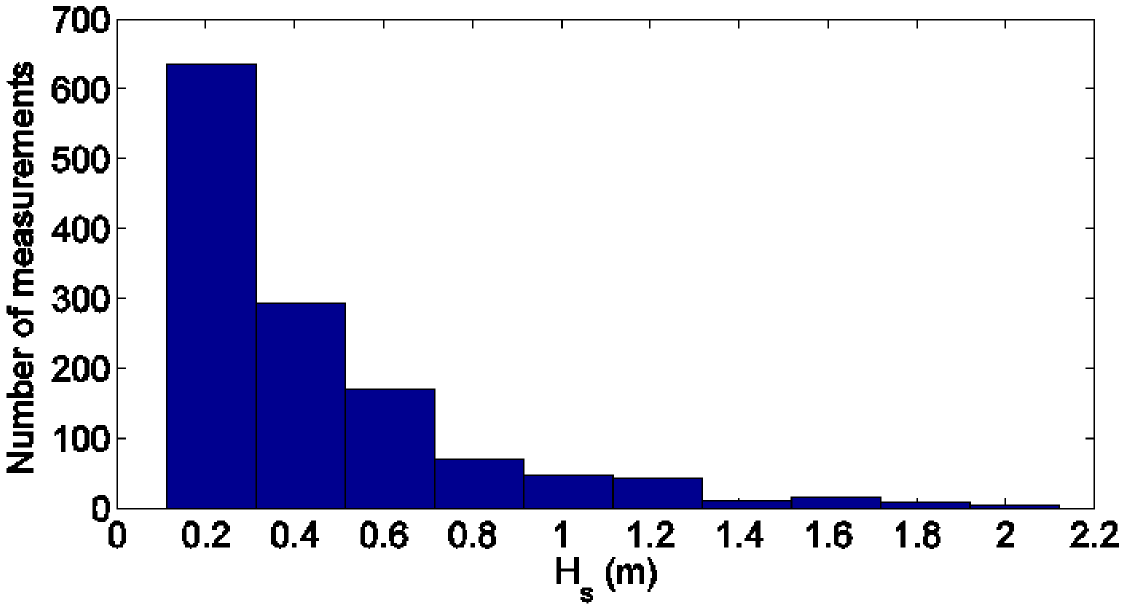


Figure 4.16 Histogram to represent the significant wave height distribution respect to the number of measurements.

The measured wave data are summarised in Figure 4.17 with the tide data (eta) recorded simultaneously. Because the tangent to the beach is approximately 108º from the north, the mean wave direction (α) was adjusted to convert these angles relative to the beach normal. This transformation results in positive angles implying easterly littoral drift and negative angles indicating littoral drift in the westerly direction. There are observed three peaks in the *Hs* above 1m and a consistent occurrence of *Hs* up to 1m the first fortnight of November. Those significantly higher *Hs* values are taking place with westerly wave directions except the data measured around the 17th and 20th November where the wave direction turns from westerly to easterly in two occasions. It is noted that the increase of *Hs* at the end of October took place during the spring tide which can effect on the tide level and cause a positive surge. In fact this is why the tidal level is higher during that period with respect to the other spring tidal cycles.

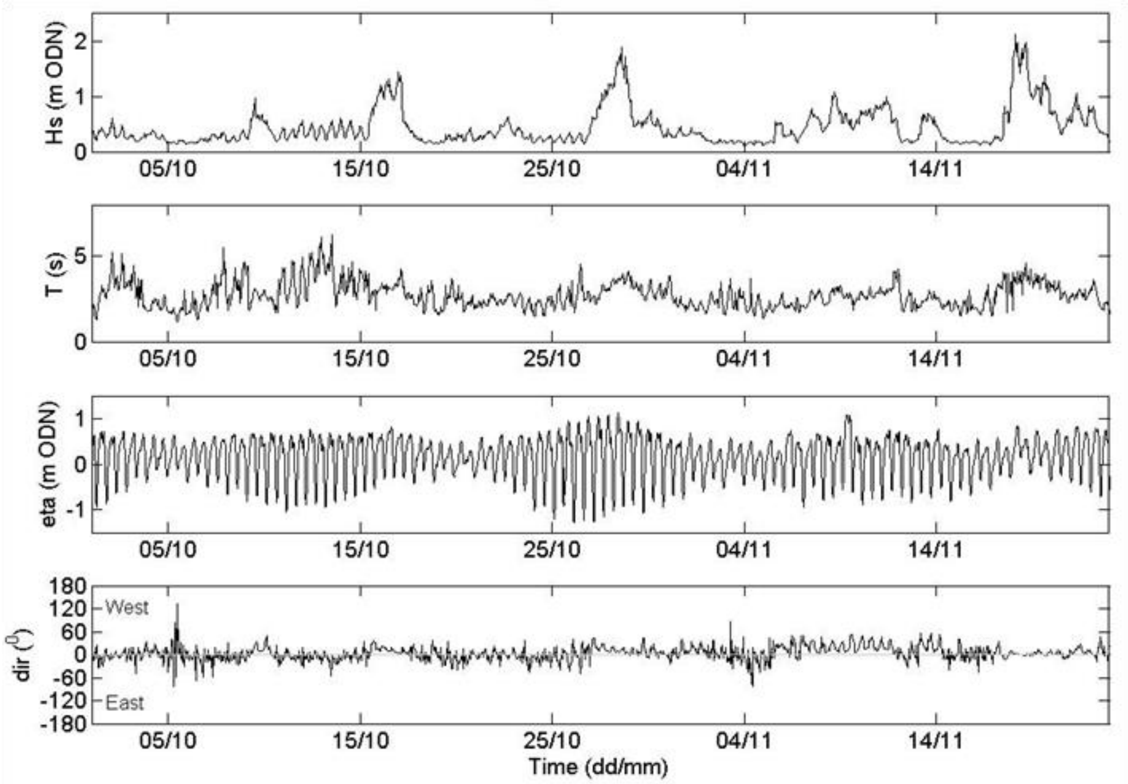
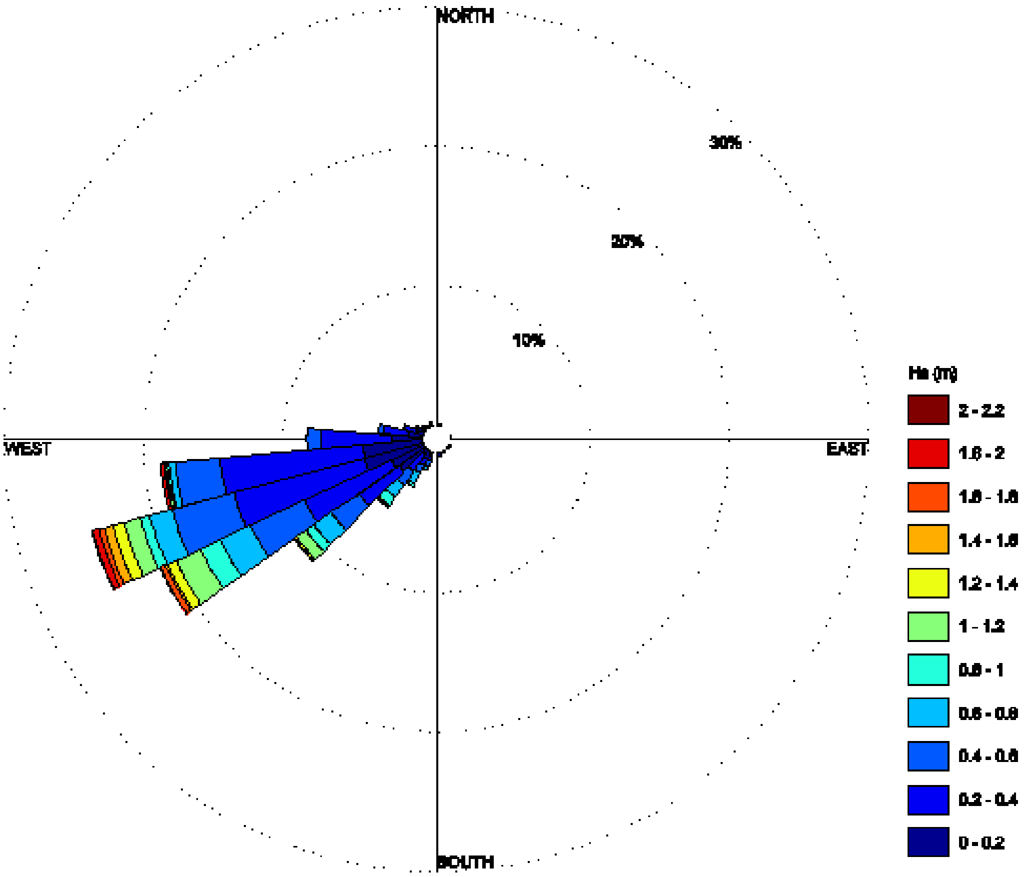


Figure 4.17 Wave parameters measured by the AWAC and tide data measured at Becton Bunny.

Figure 4.18 represents the distribution of the significant wave height and the mean wave period according to the wave angle direction measured by the AWAC with respect to the geographic north. A predominant west south west (WSW) - south west (SW) direction is observed. Although they are less frequent, the highest Hs correspond to the WSW directional sector.



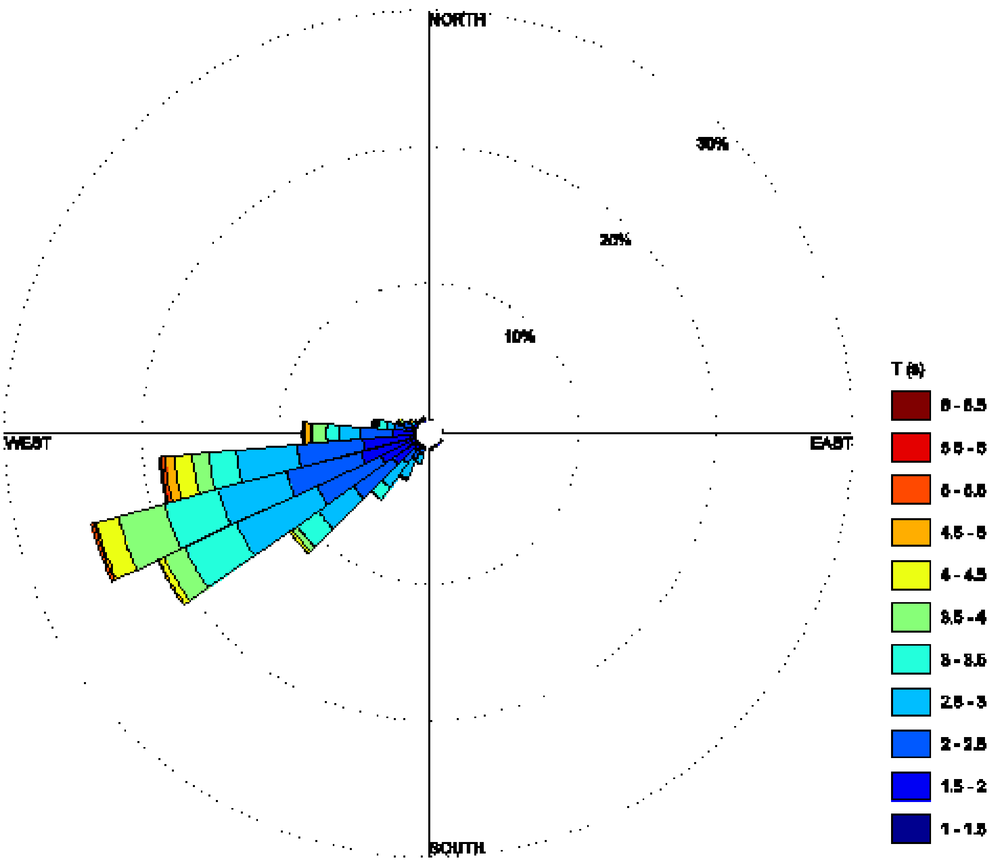


Figure 4.18 Wave rose plots for the significant wave height (top) and mean wave period (bottom) measured by the AWAC respect to the geographic north.

Due to the lack of long term wave data in the nearshore zone, an empirical approach is adopted to identify and describe storm wave conditions. Thus, based on the wave measurements there were identified three significant storm occurrences along the experiment period. Those took place the 16th October, 27th -28th October and 18th November, Figure 4.19.

The higher *Hs* records were measured during those particular days, then, because the wave energy is proportional to the square of the wave height, these events are expected to be the most energetic occurred during the experiment duration.

Figure 4.19 View of the experimental groyne from the top of the cliff (facing south) during the three major storms events identified the 16th (left) and 28th (centre) October, and the 18th November 2007 (right).

4.2.6.1 Wave transformation

Due to the AWAC being deployed in intermediate waters, at approximately 7m ODN depth, it is necessary to transform the wave data from that location to the breaker line to estimate the correct longshore wave power. Among the existing methods that explain the wave propagation from offshore to shallow waters, this study applied the Small amplitude wave theory which was developed by Airy in 1985 (also known as the Airy or Linear wave Theory). It is a simplified method to represent the wave motion and describes the change in the wave properties with depth. The equations derived from this theory were used to estimate the wave parameters in shallow waters. Then, using the wave parameters estimated at the breaker line, the longshore wave power is estimated according to the Energy Flux Method (SPM, 1984).

The longshore wave power formula requires to know the significant wave height (m), the group velocity and the wave angle approach normal to the shoreline at breaking. To obtain those parameters following the Airy Theory, it is required to know the wavelength L, which according to the full expression of the linear wave theory, for any depth h is represented by Eq. 4.1.

$$L = \frac{gT^2}{2\pi} tanh\left(\frac{2\pi h}{L}\right) \quad \text{Eq.4.1}$$

Where g is the acceleration due to gravity and T the mean wave period. This equation relates the wavelength and the wave period and is known as the dispersion equation. Due to the term L on either side of the equation, it is solved applying an iterative method, normally using as a first approximation the wavelength for deep waters *L0* derived from the linear wave theory. The wavelength was obtained using a direct solution that was derived by Hunt in 1979 (Kamphuis, 2000; Reeve et al., 2004) which is accurate to 0.1 per cent and it is represented by Eq. 4.2.

$$\frac{c^2}{gh} = [y + (1 + 0.6522y + 0.4622y^2 + 0.0864y^4 + 0.0675y^5)^{-1}]^{-1} \text{Eq. 4.2}$$

Where $y = k_0 h$being $k_0 = 2\pi/L_0$ the wave number related to the wavelength in deep water and *c* is the wave celerity that is the speed at which the wave travels. It is derived from the linear wave theory at any depth by the Eq. 4.3 $\omega = 2\pi/T$ is the wave angular frequency):

$$C = \frac{L}{T} = \frac{\omega}{k} = \frac{g}{2\pi} T\, tanh\left(\frac{2\pi h}{L}\right) \quad \text{Eq.4.3}$$

Thus the wave number, the wavelength and the wave celerity have been transformed from intermediate waters to shallow waters using the linear wave theory. Next is to estimate the wave parameters at breaking, those are the significant wave height, the water depth and the wave angle direction normal to the shoreline.

Munk (1949a), considering the Airy wave theory and the ratio $\gamma_b = H_b/h_b = 0.78$as the breaking criterion, derived an empirical formula, Eq. 4.4, to estimate the wave height at breaker point, H_b and $h_b$respectively, relating that one to the deep water wave steepness represented by H_0/L_0 (Komar, 1998). In the analysis H_0 has been assumed as the wave height measured by the instrument (even though it was deployed in intermediate waters).

$$\frac{H_b}{H_0} = \frac{1}{3.3(H_0/L_0)^{1/3}} \qquad \text{Eq.4.4}$$

The formula obtained by Munk (1949a) showed an agreement with the data especially for lower H_0/L_0values and it has been widely used to predict the wave heights at breaking from deep waters by coastal engineers (Komar, 1998). Thus, the significant breaker height, H_{sb}, has been estimated using Eq. 4.4 and hence, the breaker depth, h_b, was estimated applying the breaking criterion mentioned above.

Finally, and continuing the wave propagation calculations, another process that takes place when the wave motion is affected by the seabed is the wave refraction by which the wave propagation direction changes when waves approach the shore at an oblique incidence. Normally this process is combined with the shoaling effects consisting of the alteration of the wave height. Thus, in order to estimate the change of the wave crest direction, as it is considered to solve many practical problems, it is assumed that the seabed contours and the shoreline are approximately straight and parallel; then the changes in the direction may be related to the change in the wave celerity by the Snell's Law, Eq. 4.5 which was applied to the data to estimate the refraction to the shoreline.

$$\frac{\boldsymbol{sin\,\alpha}}{\boldsymbol{sin\,\alpha_0}} = \frac{\boldsymbol{c}}{\boldsymbol{C_0}} \qquad \text{Eq.4.5}$$

According to this relationship, as the wave approaches the coast the wave celerity diminishes and the direction of the wave incidence also decreases respect its value in deep water. Up to now, for the shoaling and refraction observations it was assumed that as the waves were propagating from offshore to inshore waters, there was no loss of energy, however, when waves pass across the transition to shallow water, they are attenuated due to the seabed friction which causes the wave energy dissipation (Reeve et al., 2004).

In order to characterise the type of breaking at Milford-on-Sea according to the measured wave data, the breaker index, ξ_b, (Iribarren Number) was estimated using the Eq. 2.11. The input parameters were the mean slope for the shingle fraction (obtained from the linear relation between the interface position and the beach elevation, Section 4.2.7.1.1), and for the wave steepness, defined as the ratio H_{bs}/L_0, the calculated significant wave height at breaking and the wavelength obtained from the dispersion equation L.

The results obtained gave a surf similarity parameter between 0.3 and 1.2. Then, according to the criteria established by Battjes in 1974 these results indicate that for values $\xi_b < 0.4$ there are spilling breaking waves whereas for values within $0.4 < \xi_b < 2.0$ the waves are plunging breaking. The spilling breaker type is characteristic of gentle beach slopes and steep incident waves, typically of sandy beaches. This type of breaking can be due to the influence of the sand section to the energy dissipation. Although only four values were estimated with a breaker index between 0.3 and 0.4, thus it can be said that Milford-on-Sea is characterised by plunging breaker waves. Those are characterised of steeper beaches, typical of shingle or gravel beaches and may be combined with flatter waves (Kamphuis, 2000; Reeve et al., 2004).

4.2.6.2 Alongshore wave power

The wave energy is important as it is the forcing mechanism by which sediment motion under the action of waves and tides. As it was described in Chapter 2, the Energy Flux method to calculate longshore transport rates, considers the immersed weight of the alongshore moving sediment proportional to the alongshore wave power per unit length of beach (Kamphuis et al. 1986). Thus, the wave transformation calculations of the wave parameters from intermediate waters to shallow waters gave us the significant wave height H_{bs}, the water depth h_b, and the wave angle approach α_b at the breaking line (subscript b).

The longshore wave power was calculated according to Eq. 2.6 and it is represented in Figure 4.20for the duration of the groyne experiment that is from the 1st October 2007 until 24th November 2007. According to the wave angle conversion normal to the shoreline, negative values of P_{ls} correspond with easterly wave angles which indicates longshore transport rates in the west direction, and positive longshore wave power corresponds with westerly wave angles what indicates easterly longshore sediment transport rates.

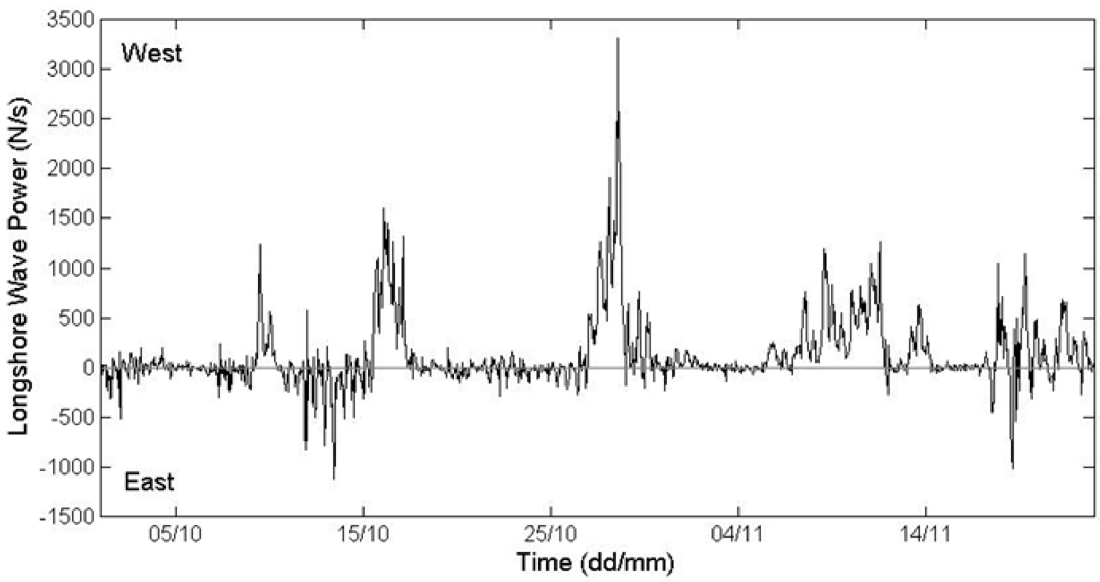


Figure 4.20 Longshore wave power at breaking for Milford-On-Sea during the experimental period.

Figure 4.21shows the comparison between the tide data measured by the tidal gauge deployed in Becton Bunny, and the wave parameters at breaking: the significant wave height and the longshore wave power, and the wave direction normal to shoreline. In the longshore wave power can be distinguished five peaks that can be related with the occurrences of higher significant wave height, in particular for the period of time corresponding with the storm events identified during the experiment (16th October, 27-28th October and 18th November). It is noted that wave direction values of proximate to 0 indicates that waves approach normal to the beach which indicates a weak or null contribution to the longshore sediment processes and the energy account is due to cross-shore processes and mainly influenced by the effect of the significant wave height.

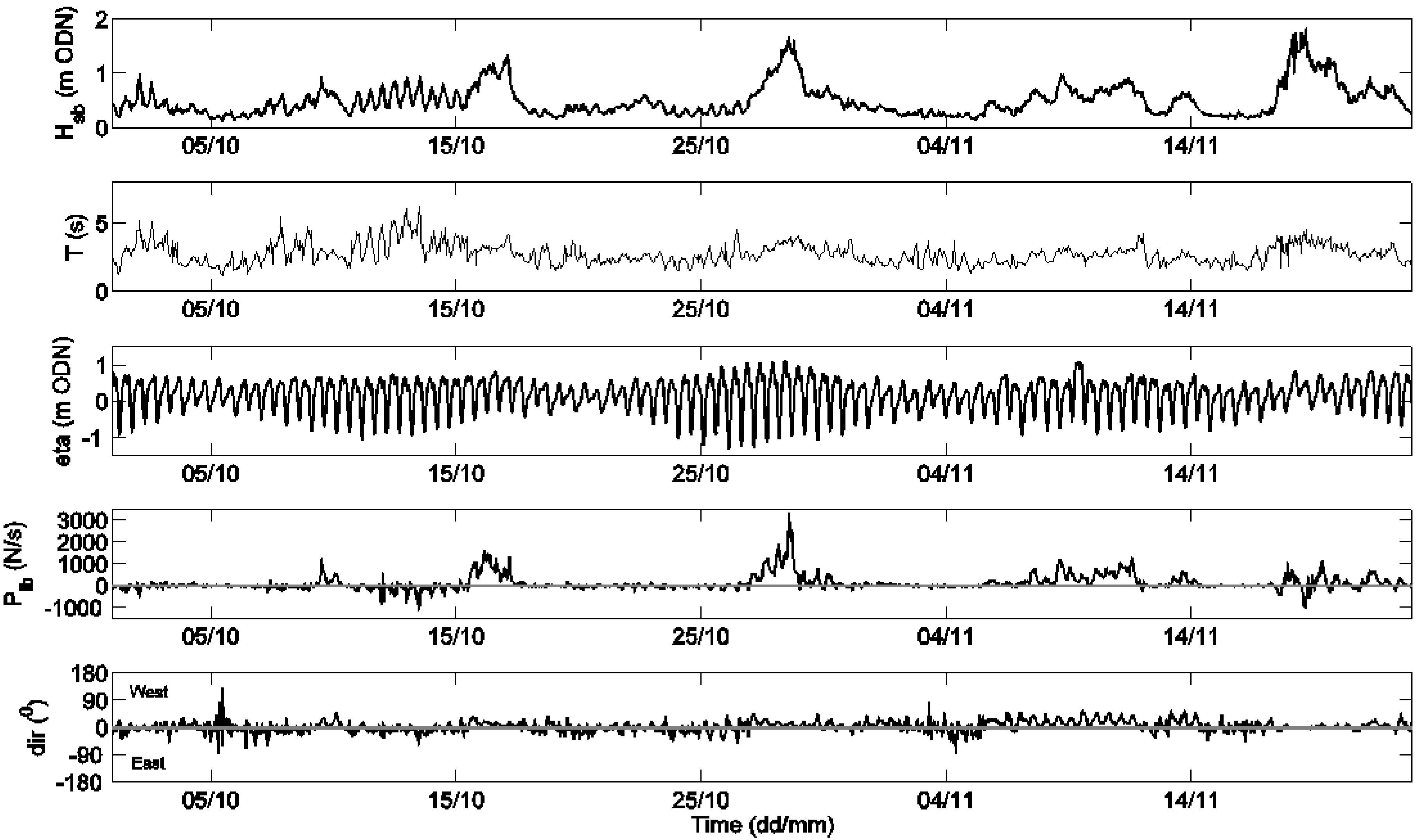


Figure 4.21 Tide data and wave parameters at breaking.

4.2.7 Beach profile surveys

The accuracy of the beach profile data has been estimated from the statistical analysis of the topographic control point measurements made at the beginning of each survey in a daily basis. According to the data, the estimated averaged errors of the RTK-GPS measurements were ±0.03 m ODN in elevation, ± 3.35 10-1 0m ODN in the Easting coordinates and ± 1.05 10-10m in the Northing coordinates. The beach profiles were surveyed at a high frequency on the cross-shore and longshore direction, about 0.5-1m interval per measurement, and 10m interval respectively.

Milford-On-Sea presents a typical beach cross- shore section characterized normally by the presence of two berms, and in occasions of three berms, a break-point step and the longshore sand bar usually below the low water spring tide level, Figure 4.22. The beach profiles are measured in northing, easting and elevation coordinates (m ODN), however, it is common practice to transform the horizontal distance (northing and easting) along the profile referred to a fixed point near the beach crest or at the toe of the cliff (Reeve et al., 2004). This horizontal distance is known as 'chainage'. Then, the transformation from northing and easting to chainage (m ODN) is calculated for all profile lines applying the least squares method to the data. By this method the chainage is estimated by minimizing the sum of the squares of the differences between the measured and baseline coordinates. The pre-established baseline is formed by those coordinates used as a reference for this chainage conversion. It is located at the toe of the cliff along the beach length and it is the starting point for each cross- shore beach line that extends seawards from that position of intersection with the baseline.

Then, setting the baseline of the survey grid in order to assess the beach profiles respect the same reference gives consistency to the analysis (Farris and List, 2007).

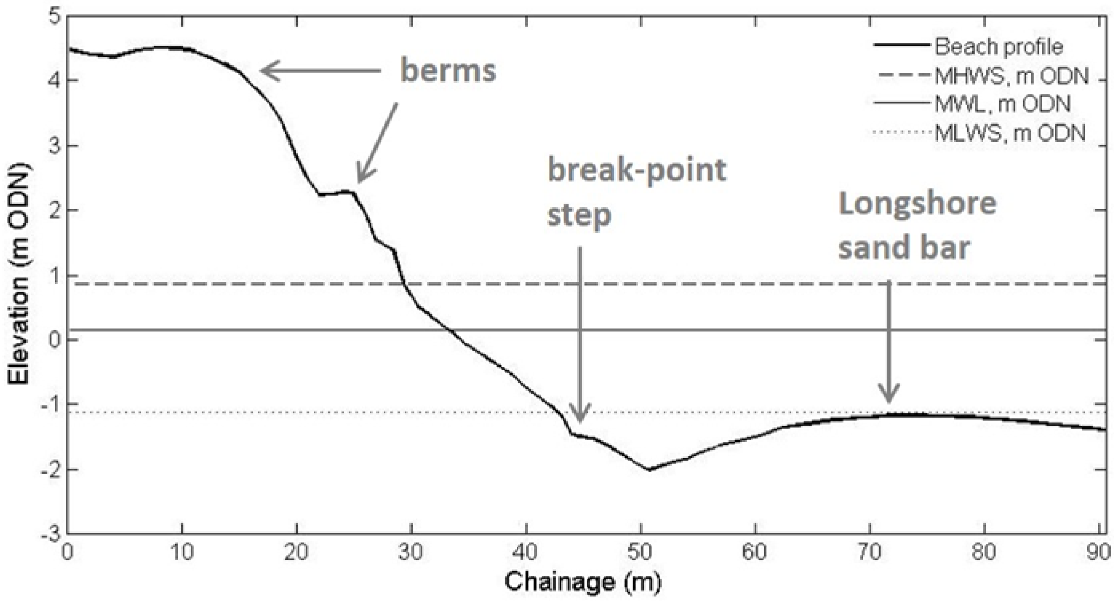


Figure 4.22 Characteristic beach profile at Milford-on-Sea. The horizontal lines indicate the water levels (mhws, mwl, mlws in m ODN).

To show examples of the profile morphology and offshore extent, the time series of four profile lines surveyed during the experiment are represented in Figure 4.23 to Figure 4.26. The initial profile surveyed corresponds to the 1st October and it is indicated by the dashed red line whereas the dashed blue line represents the profile line surveyed the 24th November when the experiment ended. Figure 4.23 and Figure 4.24 represent the profile lines surveyed at the west side of the grid, adjacent to the groyne (GW00) and an approximate 10m distance west (GW01), whileFigure 4.25 and Figure 4.26 represent the homologous at the east side (GE00, adjacent to the temporary groyne, and GE01 respectively). The variability of the beach profile is significant between the 20m and 50m chainage. Changes observed above the MHWS suggest that the range of action of the swash processes as the wave run up are important and enhance the effect of the tides. In general it is observed a tendency of accretion at the west side and erosion at the east.

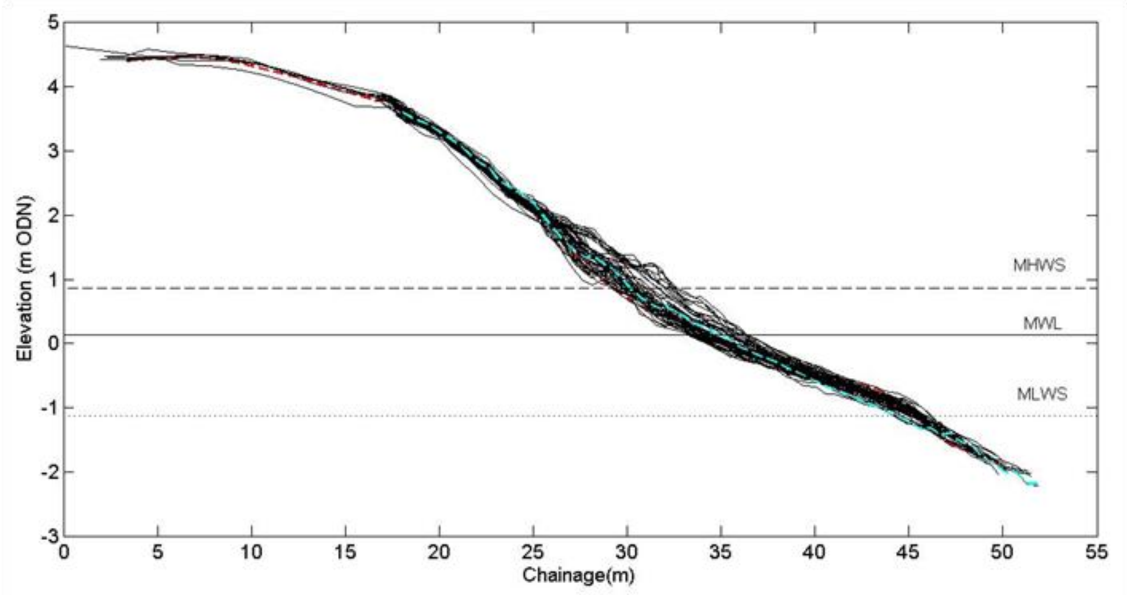


Figure 4.23 GW00 mixed survey profile; the first and last profiles correspond to the dashed red and blue lines respectively.

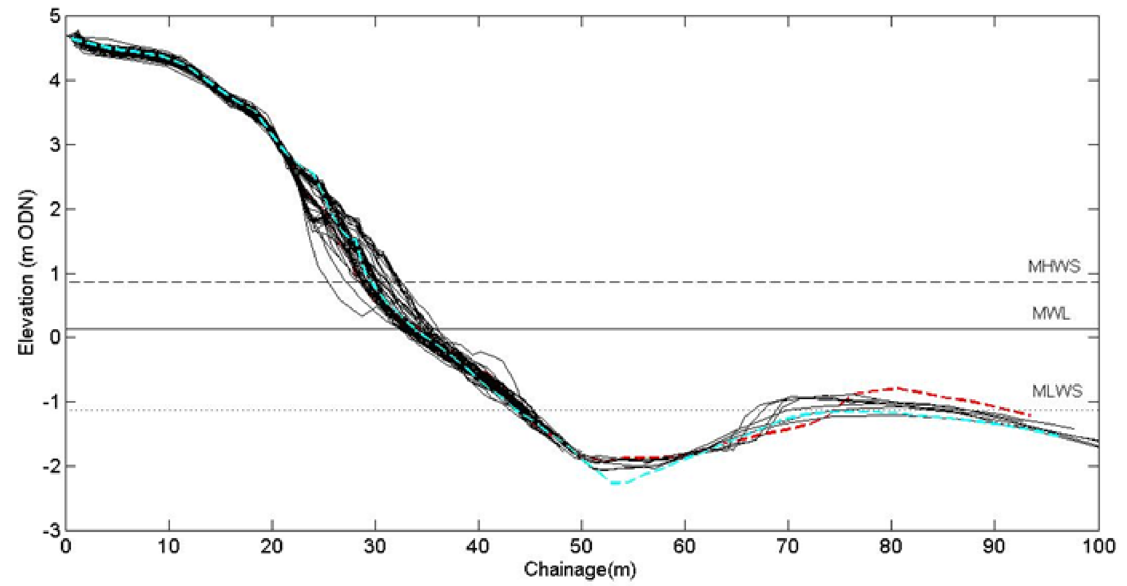


Figure 4.24 GW01 mixed survey profile; the first and last profiles correspond to the dashed red and blue lines respectively.

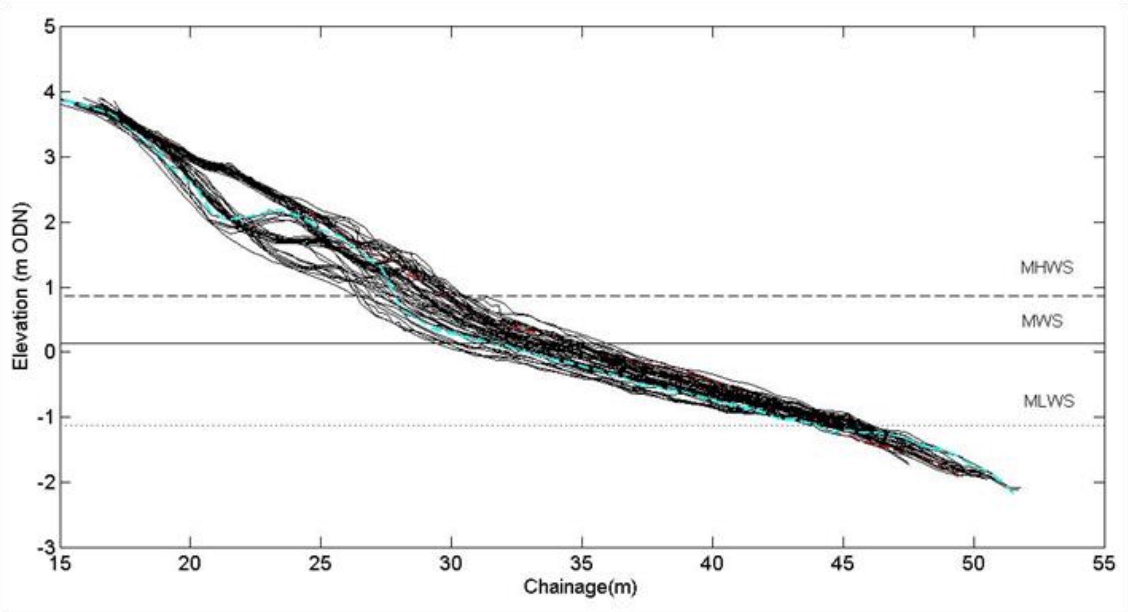


Figure 4.25 GE00 mixed survey profile; the first and last profiles correspond to the dashed red and blue lines respectively.

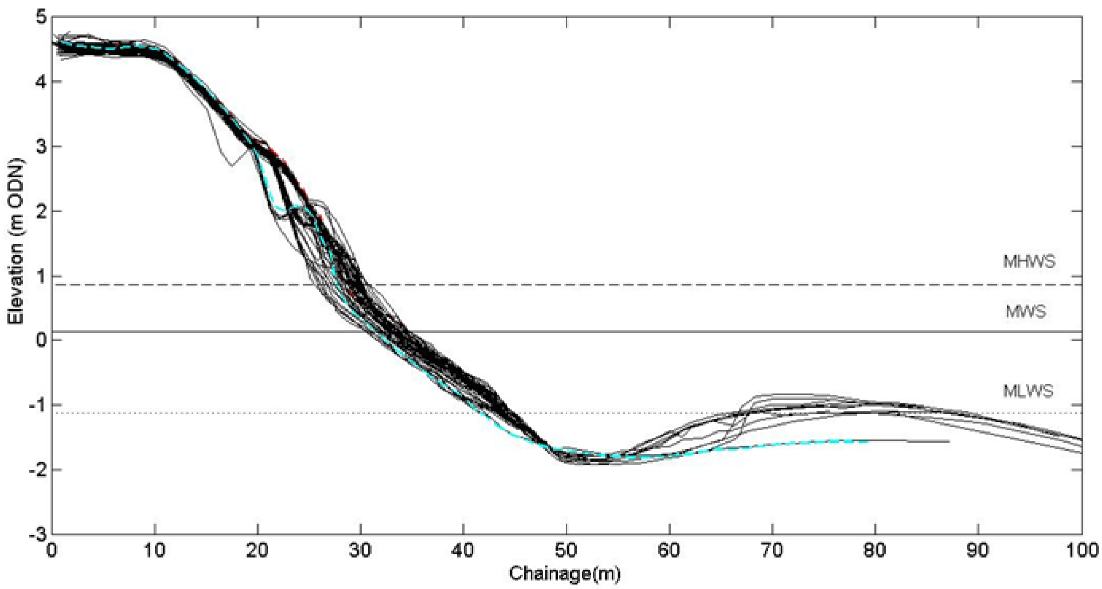


Figure 4.26 GE01 mixed survey profile; the first and last profiles correspond to the dashed red and blue lines respectively.

In the case of GW01 and GE01 it is also observed the changes in the location of the longshore bar which responds to an onshore-offshore movement.

4.2.7.1 Shingle fraction analysis

4.2.7.1.1 Determination of the shingle-sand interface

To continue the beach profile analysis further observations have been considered given the natural mixed sediments composition of the beach. The particular sediment distribution along the cross-shore section led this study to assess the shingle-sand interface behaviour and the role within the beach morphodynamics behaviour.

It was stated in Section 3.4.2.3.1 the assumption that the gravel is moving upwards and downwards on the cross-shore beach profile over a terrace of sand. To define the shingle beach profile data set, the beach profiles surveyed were examined individually, considering the feature code and validated with image data to determine the position of the interface that in turn, is the seaward limit of the shingle profile. The criteria established was that the interface position is the last shingle point measured. As an example, looking at Figure 4.27, the interface position is located at the point indicated by the yellow arrow considering that there is only sand beyond that point or, it is the last shingle point measured, rather than e.g. the sediment transition observed in the cusps.

Once the point of the interface was defined for each surveyed profile line, the cross-shore position of that shingle- sand interface related to the beach elevation was assessed. A new data set was created containing the time series of the interface position for every profile line. According to that, also the shingle section was selected in the surveyed data to create the beach shingle profile data. Unless it is noted, the primary beach profile analysis conducted in this study considers the profiles created for the shingle fraction.

Figure 4.27 Mixed sediments at Milford-on-Sea. The yellow arrow indicates the point of the interface location if sand sediments are presented just beyond that point.

The correlation analysis was conducted according to the method commented in Section 3.4.2.3.2. As a result, the regression analysis gave a polynomial equation of degree n=1 (linear) which general form, in this case, is seen Eq. 4.6.

$$y = -y_1 x = y_0 \qquad \text{Eq. 4.6}$$

y_0 is the value of the beach elevation when it intercepts the y-axis, y_1 is the slope of the line, x corresponds to the variable cross-shore distance and y is the variable beach elevation. In this case the coefficient that gives the slope of the line, y_1 indicates the foreshore beach slope resulting within a range of 1:6 and 1:8, typical of mixed and gravel beaches (Van Wellen et al., 2000).

Figure 4.28 shows an example of the linear trend existing between the gravel-sand interface positions with the beach elevation, e.g. profile line GW00. The interface is tracked along the cross-shore section within the range of 28-52m on the horizontal direction and -2.2-1.1m on the vertical beach elevation giving an interface slope with a ratio approximated to 1:7.6.

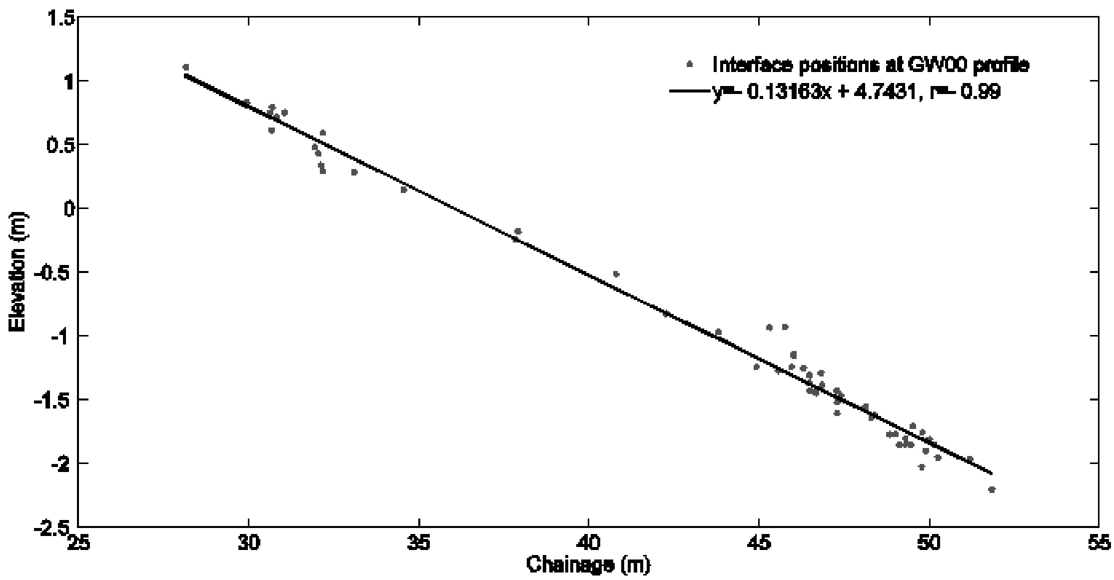


Figure 4.28 Linear relationship between beach elevation of the interface location and the cross-shore distance for the GW00 (groyne west) profile line.

The linear relationship of the interface obtained for each beach profile was set as the reference with respect to the area of the beach profiles was calculated. The estimation of the beach profile areas is explained in Section 4.2.7.1.2.

Figure 4.29 represents an example of the longshore and cross- shore interface distribution. In the top panel it is observed that the position of the interface for the 1st October (blue line) at the east side is gently retreated with respect to the western side, but in any case the distribution of the interface position at both sides of the groyne adopts a longshore alignment. The relation of this interface plan shape with changes in the elevation is shown in the bottom panel. As expected, it is seen that the beach elevation of the interface at the west side, positioned forward offshore, decreases to higher negative values than the beach elevation of the interface at the East side that is retreated.

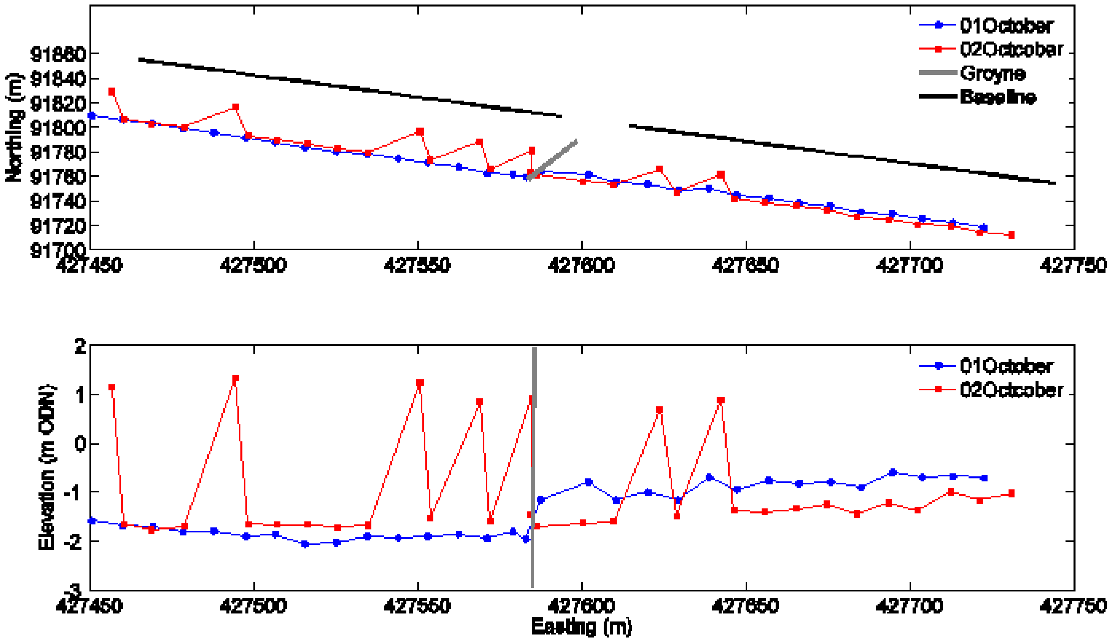


Figure 4.29 Top: represents the interface alongshore position of the profile lines surveyed at the west and east side of the groyne the 1st and 2nd October 2007 (note that there is no data for the GE15 profile line for this date). Bottom: it shows the same interface positions alongshore data points respect to their beach elevation; the black vertical line indicates the location of the groyne.

The longshore distribution of the shingle- sand interface the 2nd October is represented by the red line. The peaks indicate the presence of beach cusps as it is seen in Figure 4.30 which shows images taken the same day. The cusps affected the interface location breaking the linear configuration alongshore adopted the previous day (top panel). Thus the data points retreated with respect to the data points positioned more offshore indicate that at that position the interface is positioned at the top of the cusp, somewhere around the trough of the beach topographic feature. The distribution respect to the elevation (bottom panel) shows the points landwards at higher elevation (probably at the top of the cusps) than adjacent points that are located seawards at lower elevation. This pattern suggests the presence of cusps as it is demonstrated by the images.

Figure 4.30 Details of cusps at Milford-On-Sea formed the 02/10/2007.

4.2.7.1.2 Identification of 'Active' and 'Not active' transport

The observations presented in the previous section about the linear movement of the shingle fraction along the terrace of sand indicated by the interface shingle-sand, suggested that the tidal level has an important role in pushing the shingle up and down on the foreshore section. Thus, it was thought to relate the location of the interface with the elevation of the tidal measurements.

Being □ the tidal elevation (m, ODN), and z (m, ODN) the elevation of the mean shingle-sand interface for each day, the criteria established to identify the periods of 'Active' and 'Not active transport' was established as follows:

if $\eta \geq z$ there is 'Active transport' (tide reaches the interface position) if $\eta < z$ there is ‘Not active transport’(tide does not reach interface position)

The representative daily position of the interface at each side of the structure was estimated to relate it to the tidal data. This representative interface was calculated as the mean of the interface position of each profile alongshore.

Figure 4.31shows the elevation of the interface respect to the elevation of the tide; it is highlighted an example of the three possible scenarios: ‘No active transport’ when the tide elevation is lower than the elevation of the interface, ‘Active transport’ when the elevation of the tide is higher than the elevation of the interface, and ‘Number of hours of active transport’ when the elevation of the tide was higher than the elevation of the interface during a certain period of time. In that figure, the dotted red line represents the mean daily interface elevation at the west side of the groyne, whereas the green line represents the same at the east side of the groyne.

According to this assumption it is observed that the tide did not reach the interface during the neap tides in October. This observation can be argued because the interface location was measured always during the low tide and it is considered representative for the 24h tidal cycle while it might be possible that the tide could have reached the interface location at high tide.

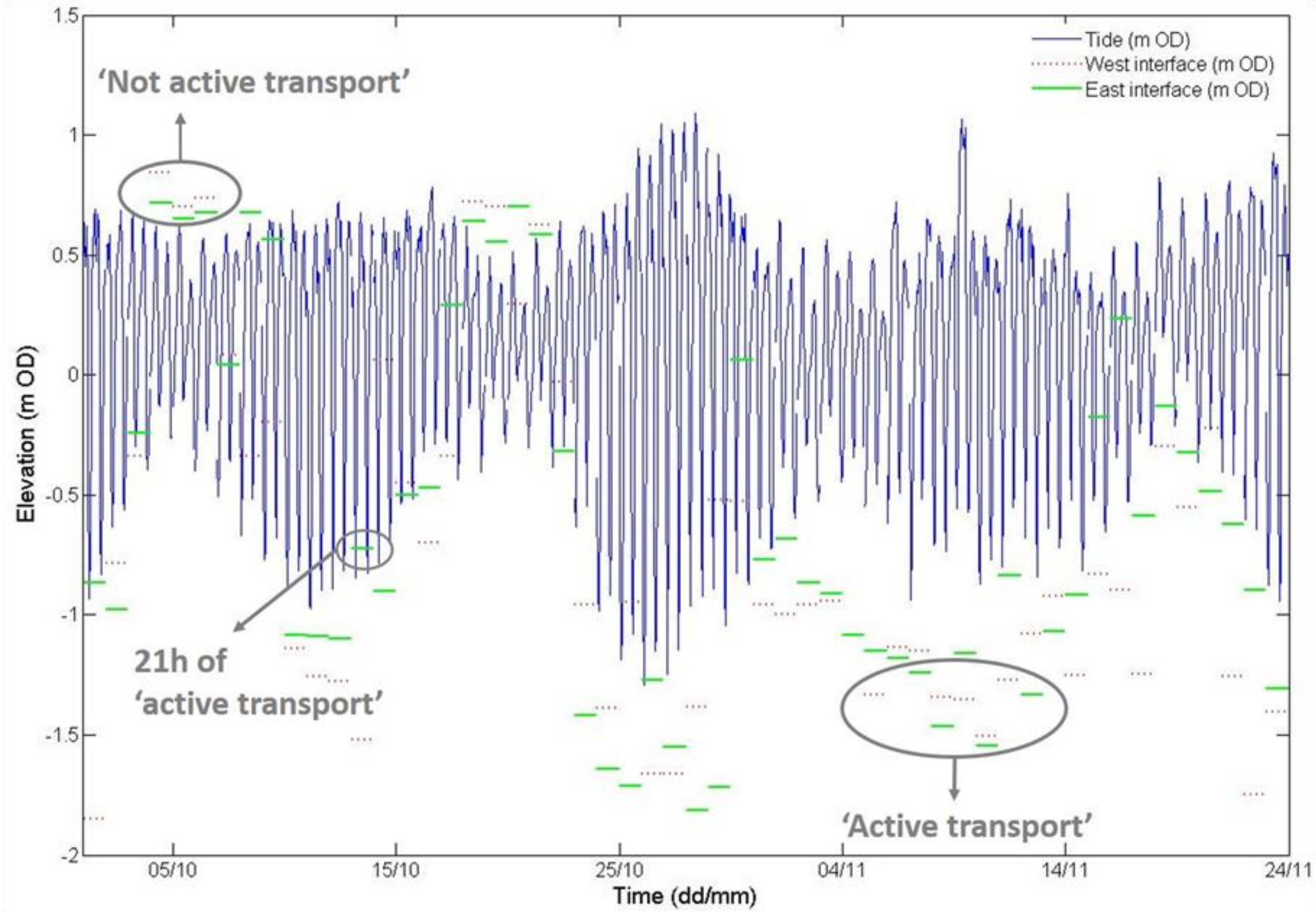


Figure 4.31 Figure of the tide measured and the elevation of the mean interface shingle-sand alongshore during the experiment time, to identify the periods of the named 'Active' and 'No active' transport.

These observations have brought some discussions whether this approach is ignoring information of beach volumes as well as if this approach is not representative of the real processes that take place over the double tidal cycle that exits at Milford-on-Sea.

4.2.7.1.3 Calculation of the beach profile areas for the shingle fraction

The linear relationship equations obtained from the significant dependency between the cross-shore distance and the elevation of the interface position were estimated for each profile line. Those equations were considered as the level of reference with respect to calculate the beach profile area. It is noted that the area used for beach volume calculations was estimated as the difference between the beach profile area and the level of reference, both up to an extent limited by the position of the interface in each case, therefore, this approach considers the beach volume above and below the level of reference (the analysis does not confine the volume above the interface).

Figure 4.32 represent an example of how the beach profile areas were estimated using e.g. the profile line GW04 (west side) the 1st October. This section was chosen as representative of the calculation as the beach profile intercepts the interface. The areas to calculate the beach volumes were estimated as follow:

- The interface positions for the profile line GW04 are fitted into the linear regression (grey points and grey line).

- The beach profile line (shingle fraction) GW04 the 1st October is plotted (dotted blue line). The linear equation derived from the interface positions is defined for that specific beach profile (dashed cyan line), this means that the seaward limit of the reference level is defined by the extent of the profile and it is termed ‘minimum level’ (dashed grey line).

- Beach profile area: The area delimited by the beach profile and the minimum level is estimated.

– Reference area: The area delimited by the interface profile (level of reference) is estimated.

– The area of interest to compute beach volumes is estimated as the difference between the beach profile area and the reference area.

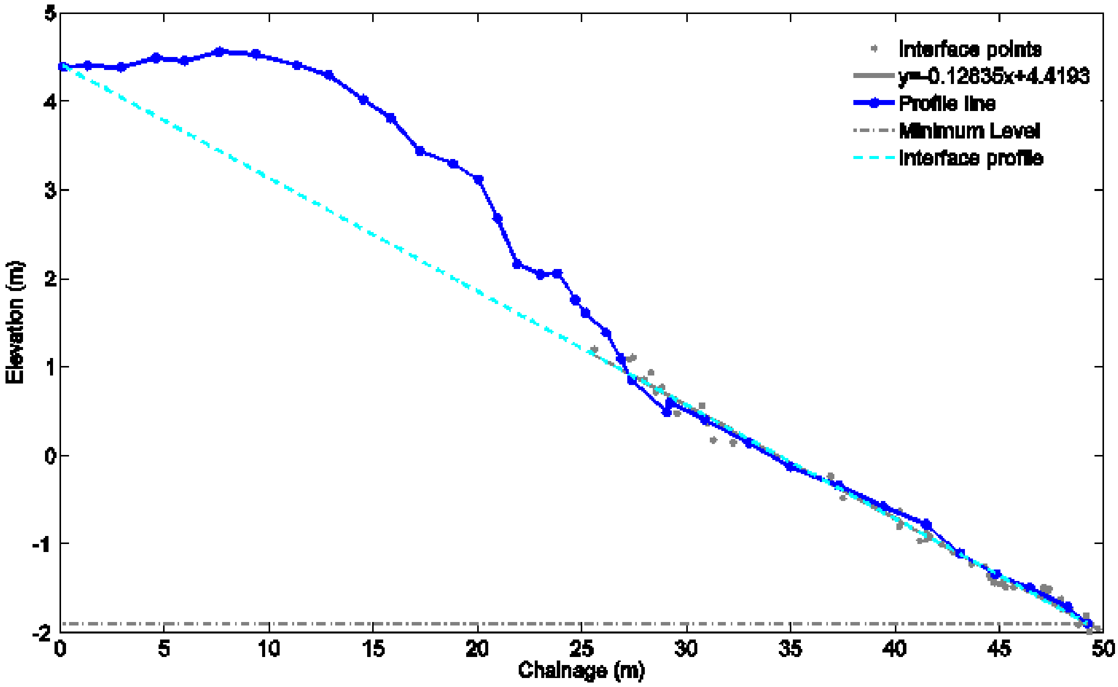


Figure 4.32 Example to show graphically the method to calculate the area difference between the beach profile (Profile line) and a level of reference that it is the linear equation that defines the interface location (Interface profile).

4.2.7.1.4 Immersed weight longshore transport rates

The beach volumes were computed according to Eq. 3.1. following the methodology presented in Section 3.4.1.1. The survey grid comprises 15 beach cells at each side of the groyne and as a result there were estimated the beach volumes for each beach cell in a daily basis. The beach volumetric difference respect to the time gives the longshore sediment transport rate between adjacent beach cells according to the method commented in Chapter 3.

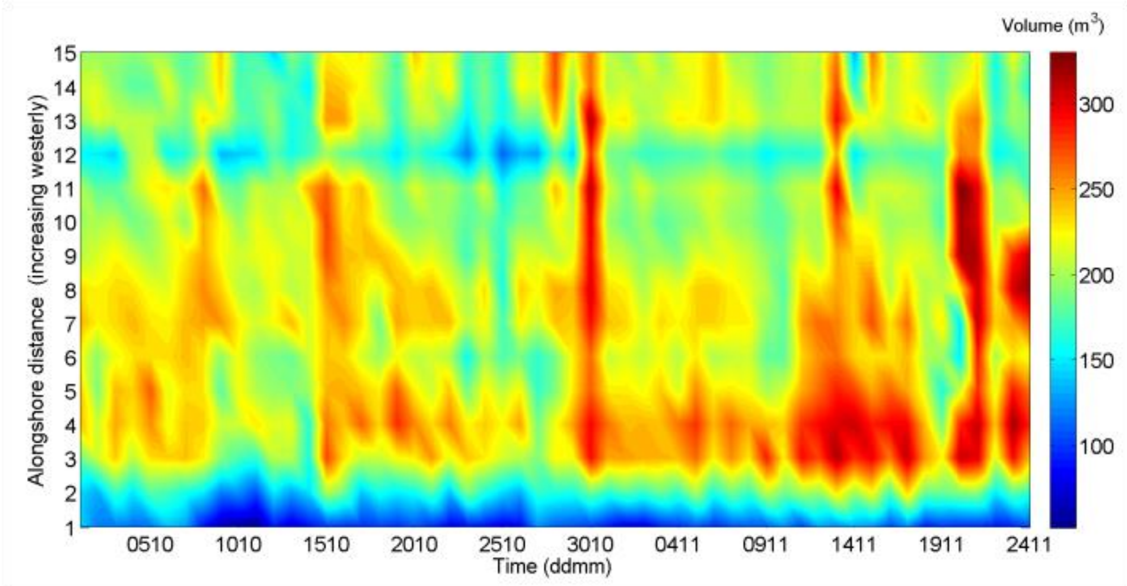


Figure 4.33 Time series of the alongshore beach volumes estimated for the shingle fraction at the west side of the groyne.

Figure 4.33shows the time series of beach volume distribution at the west of the groyne. The significant volume changes are appreciated from beach cell number 3 (y-axis) that is located approximately 30m from the structure. There is a relationship between the volume accretion occurrences with the longshore wave power corresponding the episodes of westerly wave direction approach with significant increments of beach volume. There are observed lower volume values at the beach cells 1 and 2 at the west side when comparing with the values obtained from beach cell 3. This is due to the longshore extent of the survey grid for the first two survey sections, i.e. profile lines GW01 and GW02 that were shorter than the rest of the longshore distance between surveyed lines.

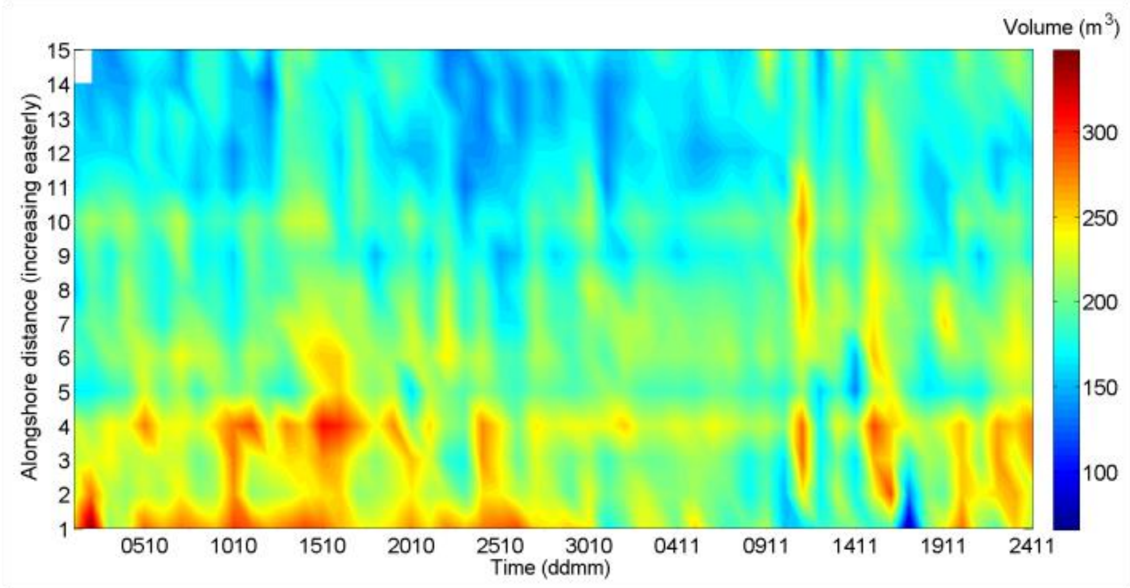


Figure 4.34 Time series of the alongshore beach volumes estimated for the shingle fraction at the east side of the groyne.

Figure 4.34 is the contour plot that represents the alongshore distribution of the estimated beach volumes at the east side of the groyne during the experimental period. The data gap observed between lines 14 and 15 is due to the absence of survey data of the GE15 profile line the 1st October. According to the observations in situ and the wave measurements there are observed sediment accretion events during the first part of the experiment, which are more noticeable approximately in the first 20-30m from the structure.

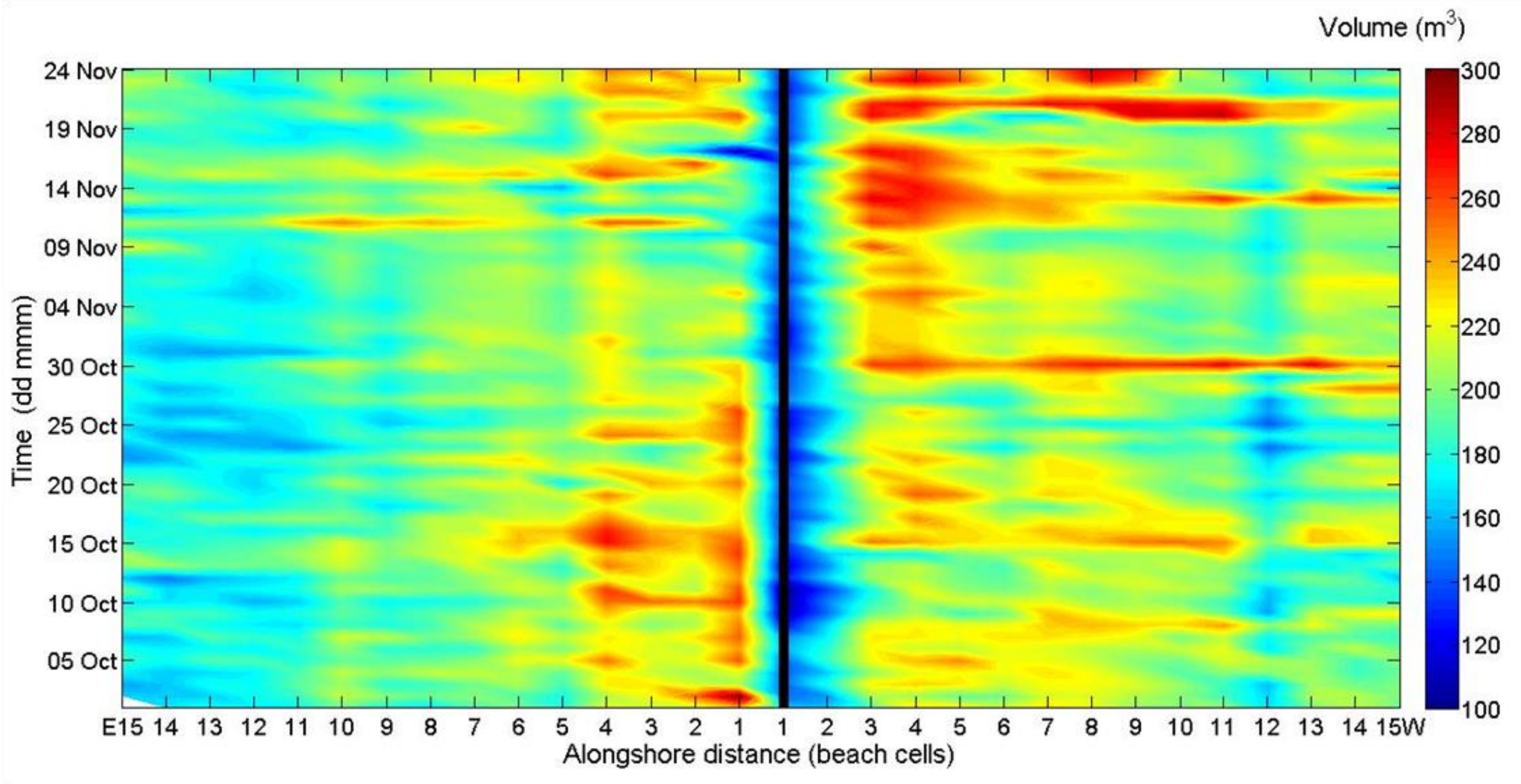


Figure 4.35 Time series of the beach volume distribution alongshore calculated using the shingle fraction profiles. The groyne location is indicated by the black line between the beach cell 1 on the left (East) and the beach cell 1 on the right (West).

Figure 4.35represents the sediment volume distribution for the shingle fraction considering the surveyed grid and during the whole experiment period. The distribution of the volume calculation correspond to the beach changes observed in situ in relation to the wave conditions. In particular, it is evidence by the groyne images presented in Appendix B.2 corresponding to the four events commented in section 4.2.8. Comparing the longshore wave power in Figure 4.20 with the volume distribution it is observed that there is an increase at the east side during the first ten days due to the easterly wave direction that it is intensified between the 10th and 15th October. In contrast, during the first two weeks of November the predominant wave direction from the west caused a significant increment of volume at the west side and no accretion and a slight erosion at the east side of the groyne. Considering the total longshore distance, at the east side the volume changes are more apparent along the 50- 60m from the structure whereas at the west side the volume changes are significant along the first 100m.

According to the methodology presented in Section 3.4.1.1., the beach volumes were calculated using Eq. 3.1., given in units of m^3 per day (beach profiles surveyed in a daily basis), and the volumetric differences according to Eq. 3.2. However, in order to include the effect of the tide elevation in the transport rates, the volumetric longshore transport rates Q were normalized by the time, in seconds, when the active transport was identified.

Then, according to the Energy Flux Method, the volumetric longshore transport rates ($Q_i(t)$), units of m^3/s, were represented in terms of the immersed weight sediment transport rates I_l, that have units of N/s as explained in Section 2.3.1 and following to the Eq. 2.2. The values of the constants used in the equation were 1025Kg/m^3 and 2700Kg/$m^3$for the sea water and sediment density respectively, 9.8m^2/s for the acceleration of gravity and a'= 0.56 as the relation of volume solids between the total volume.

4.2.7.2 Shingle fraction: total beach volume (east volume-west volume)

In this case the longshore transport rates of the shingle fraction were calculated considering the beach as a whole at each side of the groyne, this is as the summation of the volumes of the beach cells resulting in one beach volume for each day at each side of the groyne. Therefore the volumetric transport rates were estimated according to:

- West: $Q=V(t+1)-V(t)$
- East: $Q= -V(t+1)+V(t)$

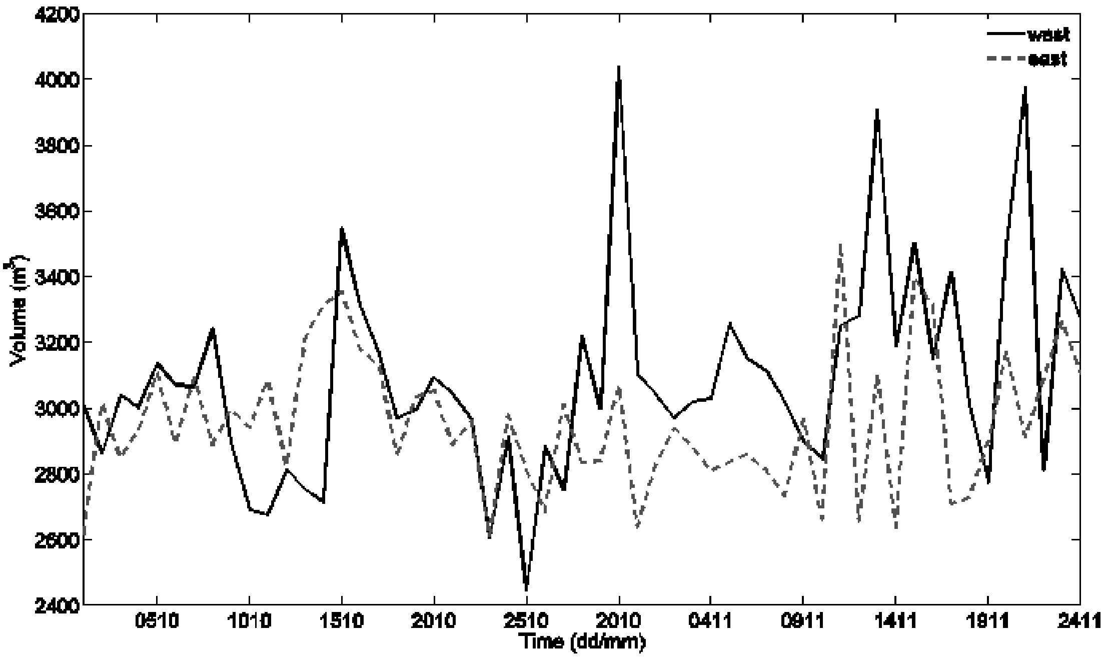


Figure 4.36 Total beach volume as the summation of the beach volume cells estimated from the shingle fraction profiles.

Figure 4.36 shows the time series of the beach volumes considering a unique beach cell at each side of the groyne. In general it is observed that the volume changes follow a similar pattern to those observed in the volume distribution when considering the surveyed grid.

4.2.7.3 Conventional mixed survey profile analysis

In order to assess the results obtained from analysing the shingle fraction of the beach profile according to the defined sediment interface it is necessary to conduct a conventional beach profile analysis for comparison. One of these traditional beach profile analyses consists in the estimation of the profile areas with respect to a specific elevation datum as a reference, e.g. the tidal levels. In this case the tidal levels chosen for the analysis are the MHWS=0.87m ODN (mean high water spring), MWL=0.14m ODN (mean water level) and the MLWS=-1.13m ODN (mean low water spring) that were introduced in section 3.2.1 (ATT, 2007). Therefore, the whole profile data surveyed is included into the analysis, i.e. the mixed, shingle and sand, cross-shore section.

At the time of writing this thesis, this analysis is currently ongoing and only the results of the analysis of the beach profiles respect to the MHWS at the west side of the groyne is presented in this thesis (Section 5.3.3).

4.2.8 Shoreline evolution from beach profile surveys (shingle fraction)

The high frequency of the measurements at which the beach profiles were surveyed, allowed a linear interpolation to be applied using the meshgrid function to create the regular space array, and the griddata function (source Matlab R2013b v8.2.0.701) to interpolate the beach elevation according to the regular grid created for represent the beach contour levels. Then, the contour levels information at a specific elevation can be extracted. In this case, the contour at 0m ODN was chosen in order to compare the shoreline measured related to the shingle fraction profiles with the shoreline predicted by the one line model (referred to 0m elevation datum ODN) that is presented in Chapter 6. According to Section 4.2.7.3, current work is ongoing investigating the shoreline changes respect to the water level contours (MHWS, MWL and MLWS) analysing two cases; one considering the profiles for the shingle fraction and the second one considering the whole data surveyed cross-shore, i.e. the ‘mixed’ profile data.

Among the experiment period, four events have been chosen characterised by the wave climate conditions to represent the distinct three storm events identified and the conditions suitable that evidence a longshore sediment transport episode (event 3). Sections 4.2.8.1 to 4.2.8.4 present the results for these beach morphological changes represented by the measured shorelines at 0m contour and those are investigated in relation with the wave conditions. The variability of the topographic contours is evidence of the groyne effectiveness. Appendix B.2 presents a comparison of the beach surface contour maps for the days assessed in this section, with an image of the beach changes observed.

4.2.8.1 Event 1: shoreline change between the 8th and the 16th October 2007

The period between the 8th and 16th October was characterised by a*Hs*= 0.48m (*Tm*= 3.33s) being the maximum *Hs* =1.32m (associated *Tm*= 3.49s) measured the 16th. Between those days the average of the mean wave direction indicates that it was predominant easterly which can induce longshore sediment transport to the west. This process can explain the shoreline advance at the east side of the groyne represented by the black and cyan beach contours (at 0m ODN) shown in Figure 4.37, with respect to the shoreline position at the west side. However on the 16th October the wave climate changed, the significant wave height increased to a maximum of 1.32m and the mean wave direction turned to westerly. The combination of high significant wave height and wave approach from the west direction in an angle to the shoreline caused a longshore sediment transport in the east direction. Those conditions can explain the contour advance at approximately the first 20m distance west to the groyne (red line) as the shoreline retreats with respect to the 14th October from about 30-40m longshore from the structure. This erosion pattern alongshore is also observed at the east side of the groyne, however, about the first 20m east to the groyne, as expected, it is noted the downdrift erosion.

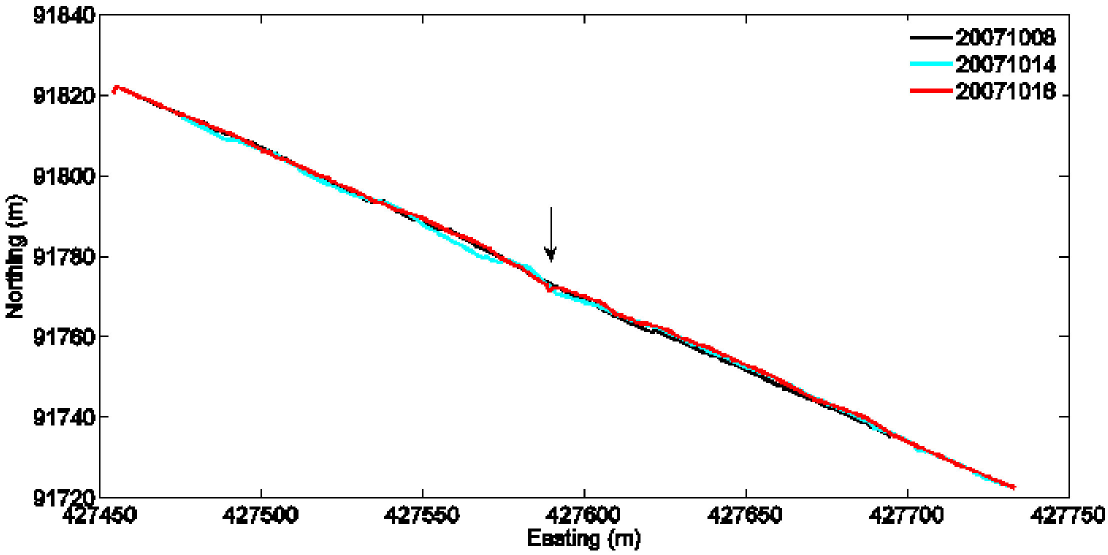


Figure 4.37 Measured shoreline at contour 0m ODN for the 8th, 14th and 16th October. The arrow indicates the location of the temporary groyne.

These shoreline changes observed at 0m contour correspond with the variability of the beach volume distribution of Figure 4.35, where an increase in volume is noted at the east side between the groyne and beach cell 5, whereas there is a decrease at the west between the 10th and 15th, to turn to increase the 16th.

The related topography contour maps are presented in the Appendix B.2.1 with an image of the beach near the groyne to evidence the change in the beach morphology.

4.2.8.2 Event 2: shoreline change between the 28th and the 31st October 2007

The period between the 28th and 31st October represent a storm – post storm occurrence characterised with a mean *Hs* = 1.43m (associated *Tm*= 4.08s) during the storm, the 28th, following by calm conditions with a *Hs* between 0.26m and 0.75m. The wave angle approach had a predominant westerly component respect to the shoreline.

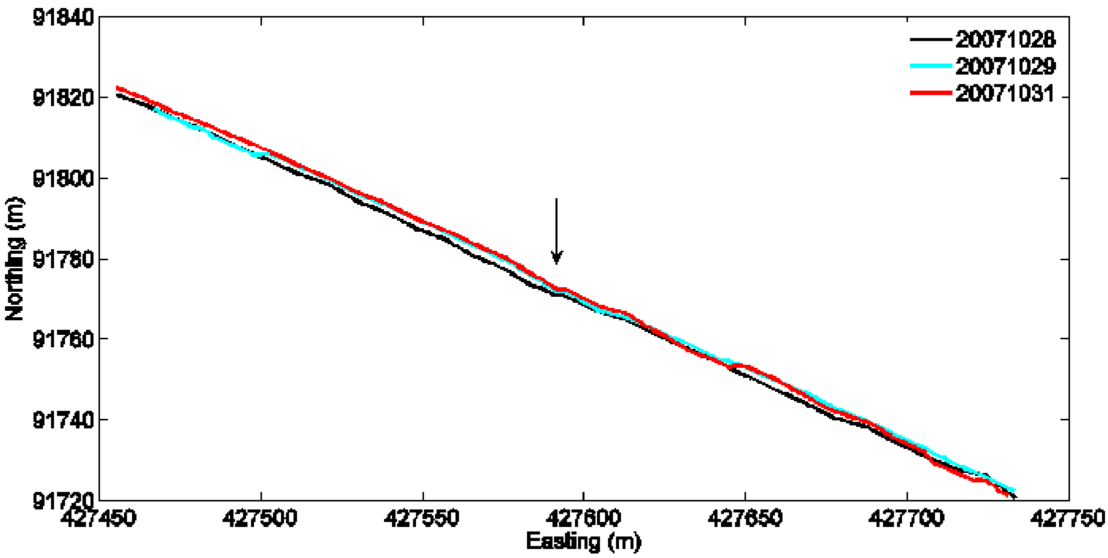


Figure 4.38 Measured shoreline at contour 0m ODN for the 28th, 29th and 31st October. The arrow indicates the location of the temporary groyne.

The corresponding shoreline contour maps are presented in the Appendix B.2.3 with an image of the beach cells adjacent to the groyne to show the change morphology.

4.2.8.3 Event 3: shoreline change between the 5th and the 8th November 2007

Figure 4.39 represents the measured shoreline at 0m contour (ODN) and it corresponds with an event of predominant longshore sediment transport with a strong westerly component of the wave direction. This caused accretion at the west side of the groyne noted by the shoreline advanced respect to the retreat at the east side. The beach images and the contour map presented in Appendix B.2.3. evidence the cause- effect of the hydrodynamic conditions on the beach morphodynamics.

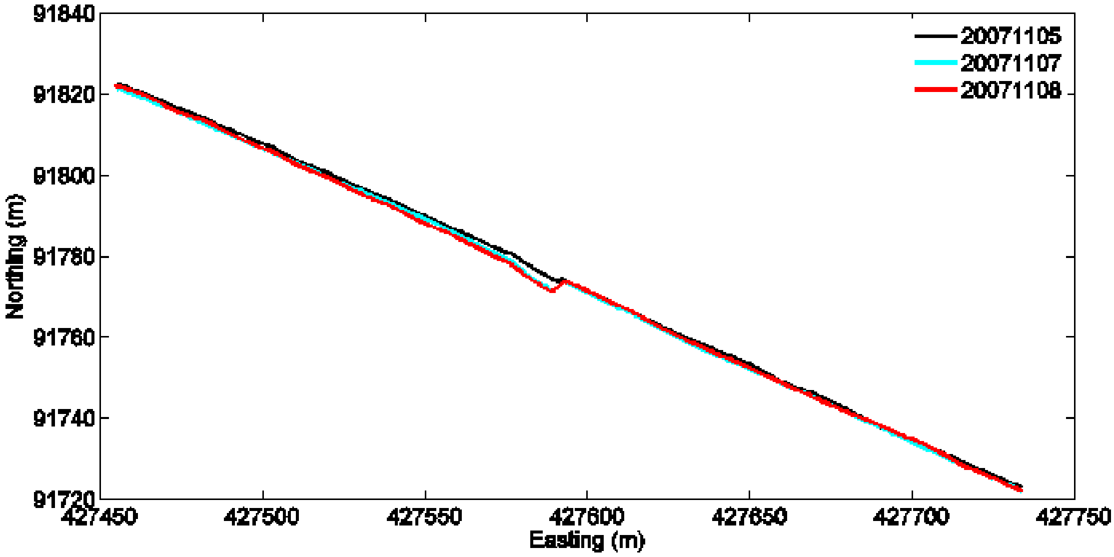


Figure 4.39 Measured shoreline at contour 0m ODN from the 5th, 7th and 8th November.

4.2.8.4 Event 4: shoreline change between the 15th and the 18th November 2007

Figure 4.40represents the shoreline measured at the 0m contour between the 15th and 18th of November corresponding to an event of calm conditions preceding an energetic storm event the 18th. The corresponding shoreline contour maps are presented in the Appendix B.2.4 with an image of the beach cells adjacent to the groyne showing the beach changes.

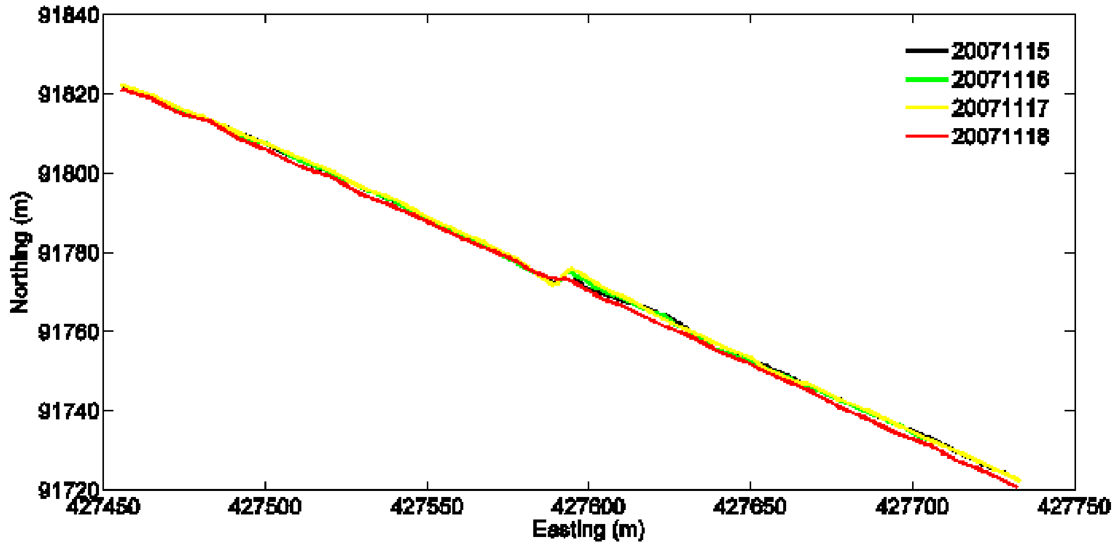


Figure 4.40 Measured shoreline at contour 0m ODN from the 15th to 18th November.

4.3 Discussion and conclusion

It is observed that longshore wave power is enhanced by the significant wave height rather than by the direction of wave angle approach respect to the shoreline however being the second one has a significant implication on the longshore transport direction. In general it was observed that the beach volume distributions correspond with the observations of the beach in situ and the volume changes are related to the hydrodynamic conditions given. Therefore it suggests the effectiveness of the Energy Flux method and the election of the impoundment technique for measure longshore sediment transport rates.

Further to the analysis of the beach profile survey data, some aspects of the survey procedures have been identified which could improve the quality of the data set, and therefore add value to the method.

The analysis of the beach profiles considering the shingle fraction may have some implications in the estimation of the beach volume due to beach features, especially the presence of cusps alongshore troughs of sand and crests of gravel. Cusps affect the sand-shingle interface position. When there are no cusps, or those are not well-defined, there used to show a longshore sand-mixed area about 5m width between the onshore limit of the swash action and the offshore limit of the gravel slope.

Site observations during the experiment have demonstrated the capability of the temporary groyne for trapping the sediments alongshore. Field observation suggested a link between the sediment deposition and /or erosion with the wave conditions.

According to these characteristics, Milford-on-Sea may further be identified as a composite mixed beach (Mason et al., 1997) that is analogous to the "composite gravel beach" according to the morphodynamic model proposed by Jennings et al. (2002) based on the study of 42 gravel beaches of the South Island, New Zealand.

The beach profile measurements allowed us to track the movement of the gravel interface under the action of waves and tides and relate those to the tidal cycle. It was found out that the position of the interface gravel-sand has a linear relation with the position on the beach elevation and on the cross-shore distance giving a foreshore beach slope within the range 1:6 and 1:8, typical of mixed and gravel beaches (Van Wellen et al., 2000). Thus, at this site it is assumed that the shingle upper beach appears to determine the performance of the beach as a defence and, furthermore, that the sand and shingle appear to behave as two distinct morphological systems, with the shingle overlaying a compacted and stable horizon of sand.

Chapter 5

5 Calibration of the CERC Equation at Milford-on-Sea

5.1 Introduction

This chapter presents the longshore sediment transport coefficients obtained when the immersed longshore sediment transport rates are related to the longshore wave power through the CERC Equation (1984). In chapter 4 the I_l were obtained for the case of the shingle fraction and at both sides of the groyne, and in turn, considering the scenario of survey grid and the scenario of the whole beach; and the I_l obtained for the case of the mixed profile and the MHWS (0.87m ODN) at the west side only considering the case of the survey grid. As a result, there were obtained four coefficients for the case of the shingle fraction and one coefficient for the mixed profile case.

5.2 Methodology to estimate the sediment transport coefficient k

According to the Energy Flux method and following the CERC Equation (1984) the longshore sediment transport coefficients were calculated using the Eq. 2.1. In order to do that and as it was explained in Chapter 4, the immersed longshore transport rates I_l were calculated using Eq. 2.2 and the longshore wave power was estimated applying the Eq. 2.6.

Overall, there is an immersed longshore transport rate I_l measurement per day and per beach cell whereas there are longshore wave power P_{ls} estimations per hour and for the surveyed beach grid as a whole.

Then, in the case of considering the survey grid or the unique beach cell, the calibration has been conducted as follows: one k coefficient per hour and per beach cell was calculated and then the average over the 24 hours was calculated. Then there is estimated a mean *k* coefficient per day and per beach cell. The next step was to calculate the average alongshore for each day; and finally the *k* coefficient at each side of the groyne is calculated as the average of the longshore mean transport coefficients.

5.3 Results: sediment transport coefficient *k*

5.3.1 Case 1: Shingle fraction and survey grid

In terms of immersed weight rates, there was calculated one longshore sediment transport rate per day and per beach unit (normalised by the time in seconds that there was determined the 'Active transport' and 'No active transport'). As the wave data were recorded at hourly intervals, the longshore wave power was also estimated at every hour. Thus, when applying the Eq. 2.1 to the data to obtain the *k* sediment transport coefficient, there is obtained one *k* coefficient per beach cell and per hour for each day (k_i (t) given by [54days x 24hours] data matrix, i= number of beach cell). The mean of the hourly data was estimated as the daily *k* for each beach cell, then, a daily coefficient alongshore was estimated as the mean of the *k* coefficients on the longshore distance and it was named $\overline{k_l}(t)$. The final *k* coefficient at each side of the groyne was estimated as the mean of the alongshore mean coefficient $\overline{k_l}(t)$, thus it resulted in two *k* values. Using the significant wave height at breaking, the sediment transport coefficient at the west side of the groyne is k_{west} = 0.1508 and the sediment transport coefficient at the east side of the groyne is k_{east} = 0.2579.

Figure 5.1 shows the time series of the alongshore mean sediment transport coefficients, $\overline{k_l}(t)$, at both sides of the groyne. As the estimated longshore sediment transport rates are subject to the effect of the tide, the data gaps observed (e.g. 4, 5 and 6th October) correspond to the identification of 'No active transport' period between. There are observed some oddities in the data time series characterised by the peaks at particular dates. Those anomalies are investigated quantitatively by the estimation of the standard deviation, and qualitatively by evaluating the variability of the wave climate conditions and beach morphology at that time.

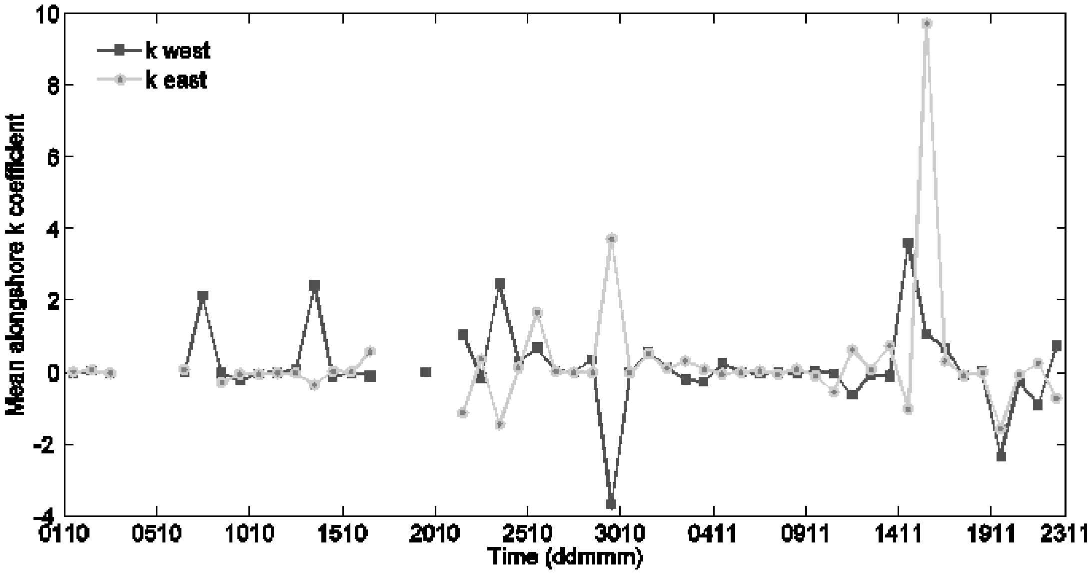


Figure 5.1 Alongshore mean sediment transport coefficient k.

The standard deviation, σ^2, of the alongshore mean *k* coefficient values at each side of the groyne was estimated to assess the degree of dispersion of the sediment transport coefficients respect to the mean. The standard deviation at the west side is 1.0650 and at the east is 1.6196. This indicates that the sediment transport coefficient is not constant on the longshore direction either in time, and it varies more than one order of magnitude respect to the calculated alongshore mean k.

In general it is observed a similar pattern of the *k* coefficients at both sides of the groyne, however it is noted that the tendency is opposite at specific events. Indeed, those events are identified as anomalies as the values deviate significantly from the mean. Those anomalies of sediment transport *k* coefficient at the west side of the groyne were the 8th, 14th, 22nd, 24th, 30th (significant peak < 0) of October and the 15th, 16th and 20th November. Whereas the anomalies of the sediment transport *k* coefficient at the east side were observed the 22nd, 24th and 30th (significant peak >0) of October and the 15th, 16th and 20th (significant peak >0) of November. It is remarkable that for those dates the coefficients have an opposite behaviour except for the 20th November and also they occurred when there were energetic wave conditions.

It is also noted the result of some negative longshore sediment transport coefficients and by definition, a constant of proportionality, that it could not be possible. This suggests a revision of the analysis since the calculation of the immersed longshore sediment transport rates as well as the longshore wave power. Also it may indicate that other hydrodynamic processes may be taking place and have an important effect, e.g. tidal currents or cross-shore sediment transport, and the method used may not be the appropriate for considering those physical processes.

5.3.2 Case 2: Shingle fraction and one beach unit

In this case the beach volume at each side of the groyne was calculated as the summation of the beach volume cells for each day, as a result, there was a daily beach volume at each side of the groyne.

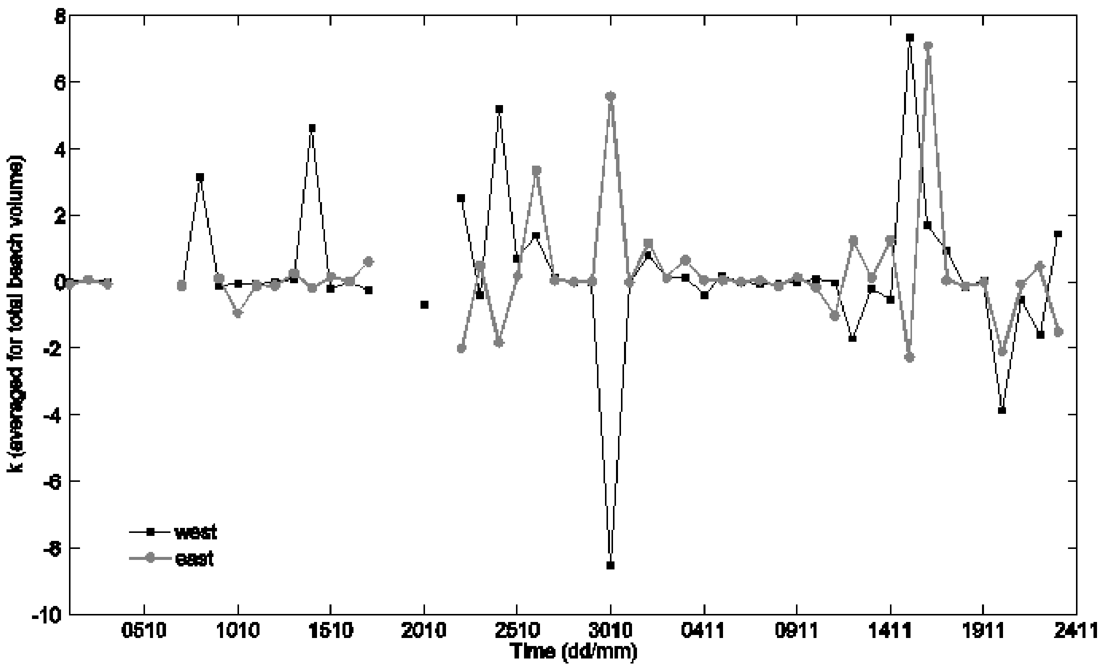


Figure 5.2 Alongshore sediment transport coefficient *k* considering the summation of the beach volume cells.

The longshore sediment transport coefficient *k* was obtained at both sides of the groyne, however the standard deviations differ and it is significantly higher at the east side. The average of the *k* coefficients obtained at the west side of the groyne was k_{2west} =0.2250 (subscript 2 refers to case 2) with a standard deviation of 2.163, and at the east side of the groyne the result was k_{2east}= 0.2265 with a standard deviation of 6.986.

In general it is observed that the longshore sediment transport coefficients have a similar pattern to that one obtained in case 1, however the difference in value is less than the double higher when in case 2, specially for the *k* coefficient at the west side.

5.3.3 Case 3: Mixed beach profile, MHWS and survey grid

This section presents the longshore sediment transport coefficient obtained from the analysis of the beach profile data considering the mixed, sand and shingle, surveyed data and estimating the profile areas respect to a sea water level, in this case the MHWS (0.87m ODN) and at the west side of the groyne. The longshore sediment transport coefficients were obtained as a result of relate the immersed longshore sediment transport rates with the longshore wave power. This coefficient was obtained as the average of the *k* longshore sediment transport coefficient and the result was kmixed MHWS = 0.067.

Comparing this value with those presented in Figure 5.2 it can be said that it is in agreement in the order of magnitude, however it is slightly higher than the 0.058 obtained by Nicholls and Wright (1991) also using *Hsb* and the 0.054 used by Ruiz de Alegría-Arzaburu (2010) and previously used by Chadwick et al. (2005).

5.3.4 Comparison between case 1, case 2 and case 3

Considering the immersed longshore transport rate for the shingle fraction, two *k* coefficients were obtained in case 1 and case 2. In general, those obtained considering the beach volumes as the summation of the beach volume cells (case 2) are higher than those obtained in case 1, this is considering the volumetric differences between adjacent beach cells at both sides of the temporary structure.

Figure 5.3 represent the comparison between the alongshore mean longshore sediment transport coefficients as a result of the analysis of the shingle fraction. It is observed a similar pattern for the both cases, however the values obtained in case 2 are higher than those obtained in case 1. Understanding the *k* coefficient as a constant of proportionality, it may suggest that for the same longshore wave power the immersed longshore transport rates considering the unique beach cell is overestimated respect to those obtained considering the surveyed grid, especially for the more energetic events.

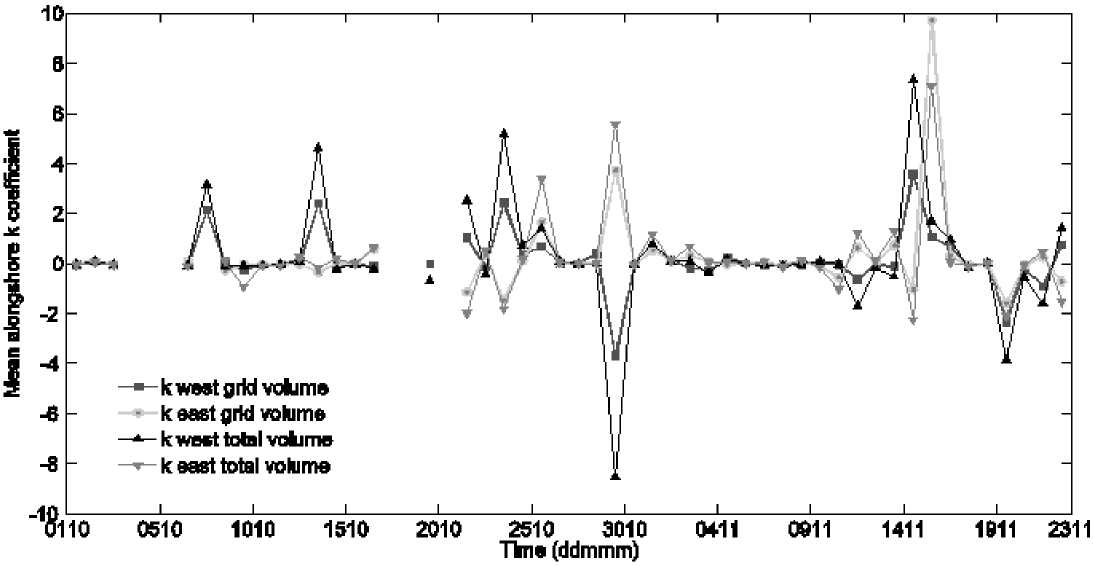


Figure 5.3 *k* coefficient time series case 1(surveyed grid) and case 2 (unique beach cell).

The *k* coefficient obtained through the conventional analysis of the beach profiles considering the MHWS at the west side is in general, of the same order of magnitude of those obtained in the literature for coarse sediments and using significant wave height.

5.4 Discussion and Conclusion

The transport rate coefficient *k* it is understood as a coefficient of proportionality of the immersed sediment transport rate (N/s) respect to the alongshore wave power (N/s).

Here, it was calculated applying the CERC Equation (1984) that presents the advantage of calculating a dimensionless k.

The calibration of the CERC Equation resulted in two different *k* coefficient values, *k*= 0.1508 at the west side of the groyne, and *k*=0.2579 at the east side, calculated as the mean *k* alongshore and using significant wave height when considering the shingle fraction and the surveyed grid. Those values are one order of magnitude higher than those noted in the literature. Also, the *k* coefficients obtained are different at both sides of the groyne. To assess these discrepancies the following are investigated:

- The influence of the slope for the shingle fraction:
    - It was observed that the slope of the shingle fraction varies alongshore and consequently it may affect the extent of the profile.
    - According to Kamphuis (1986), the beach slope is also related to the tidal fluctuations in a way that the beach slope becomes steeper on a concave beach profile as far as the tide level rises what causes much major sediment transport rates. This may explain why the east side of the groyne that is slightly steeper than the west side present a higher value.
- Calculation of the longshore sediment transport rates: considering the beach cells defined at both sides of the groyne according to the survey grid. Given the high frequency of the beach cells, it is thought that this method provides more accuracy in the results than the assumption of a single beach cell.

The *k* coefficient obtained from the conventional beach profile analysis suggested that the inclusion of the effect of the tide by calculating the beach volumes respect to the tidal levels and the consideration of the whole range of profile data surveyed may give better approximations to k values noted in the literature than those values obtained considering the shingle fraction and the ‘Active transport’ assumption.

It is also thought whether the application of different method, e.g. numerical modelling, to calculate the wave parameters at breaking for the longshore wave power would give a better approximation. The integration of the longshore wave power per day was investigated but it was not representative of the hydrodynamic processes that occurred.

Chapter 6

6 One-line modelling at Milford-on-Sea

6.1 One-line model by Dr.Baoxing Wang

A one-line modelling framework coupled with a Refraction- Diffraction wave computation model (Li, 1994) is used to predict the shoreline response to the groyne being placed at the site. This approach allows the beach-wave interaction to be captured in the simulations. The model of the shoreline movement includes both cross-shore and longshore sediment transport. The method of lines with an adaptive time step technique is used to solve this equation. The wave model is driven by offshore wave data, and is solved to provide the wave height, bearing and water depth at the breaking line. This information is then used to calculate the sediment transport rate variation along the section of coastline, using the CERC formula. Results are presented comparing the model output and the observed shoreline response from field surveys.

6.2 Application for Milford-on-Sea

For this study a one line model developed by Dr.Baoxing Wang and greatly modified by Dr. Jason Hughes for its applicability for Milford-on-Sea, was ran to test the empirical *k* transport coefficients obtained, k_{west} = 0.1508 and k_{east}= 0.2579.

The recorded wave height measurements by the AWAC, which was taken approximately 600m offshore, and tide elevation data measured at Becton Bunny tidal gauge were input into the model that update the information in hourly time steps.

6.2.1 Model Inputs

The bathymetry measured before the experiment, the 28thAugust 2007, was input into the model. The wave model is solved for the initial bathymetry on each time step. In the simulation, the water depth is updated according to the measured water level at that time. Then, the model calculates the initial shoreline at 1m contour level from the bathymetry which is updated on each time step of the wave model computation. The mean sediment size considered was 10cm, a fall velocity of 0.055 (Kamphuis, 2000), porosity value for gravel of 0.4 (ratio volume solids/total volume is 0.6), and the mass density of sand considered is 2,650 Kgm^{-3} (SPM, 1984).

The simulations were carried out on a grid which is rotated from the usual Northing/Easting system of coordinates, thus the grid was aligned to the general direction of the shoreline in the one line model simulations. The rotation used was 19.05 degrees in an anti-clockwise direction from the Northing/Easting grid. The wave bearings at the breaking point are therefore in this rotated coordinate system. Thus, the data is only generated at times at which the wave heights were exceeding the threshold of 0.25m as it does not converge for low wave heights. This can be assumed as there is very little movement in the line predicted by the one line model when the wave heights are small.

6.2.2 Model outputs

In order to assess the shoreline changes the outputs of the model provide the initial shoreline calculated from the bathymetry and the shoreline predicted in an hour time step for significant wave height higher than 0.2m.

Figure 6.1 represents an example of the capability of the model. In this case the shorelines were predicted using the k = 0.1508. As higher the k coefficient the model will over-predict the shoreline changes with respect to the initial shoreline as it is shown in this example. It is also observed that the effect of the groyne is notable at the location closer to the structure as expected. Qualitatively the tendency predicted by the model is in agreement with the measurements however quantitatively the difference in the shift on the offshore direction is higher than the expected change.

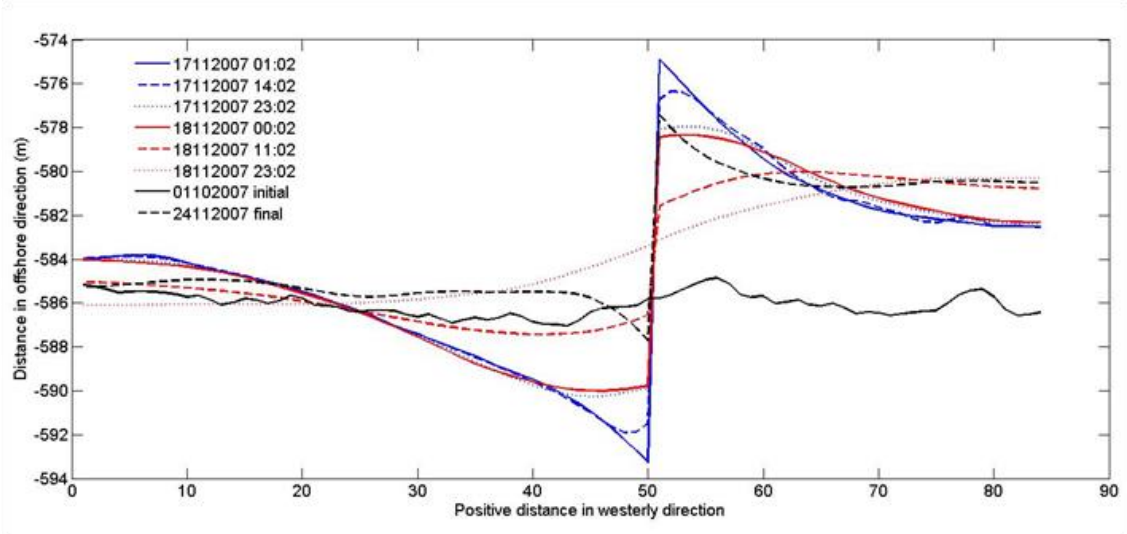


Figure 6.1 Example of model predictions for the gravel fraction on Milford-On-Sea for *k* = 0.1508. Shoreline change during the storm event occurred the 17th and 18th of November (full, dashed and dotted red and blue lines respectively); and initial and final (full and dashed black lines) shoreline positions. The groyne is located at x=50 in the horizontal direction divided in units of 2.5m, and the y-axis extends in the positive direction offshore.

6.2.3 Comparison between measured and modelled shorelines

In order to compare the shoreline obtained from the beach surveyed data with the shoreline predicted by the model, it is necessary to transform and rotate the measured Easting and Northing to match the model coordinates. First of all, the initial measured shoreline that corresponds to the 1st October given in Easting (m) and Northing (m) is considered; those coordinates are de-mean in order to set the location of the groyne at (0,0), this gives the longshore distance in negative and positives being positive in the westerly direction. The de-mean data are fit into a linear relation to find the angle of rotation, θ, given by the arctangent of the slope of the linear equation rotated 90º respect to the origin. Thus, the rotation matrix, Eq. 6.1.is applied to the de-mean data to obtained the transformed and rotated shoreline.

$$\begin{bmatrix} x \\ y \end{bmatrix} = \begin{bmatrix} cos\,\theta & -sin\,\theta \\ sin\,\theta & cos\,\theta \end{bmatrix} \begin{bmatrix} X \\ Y \end{bmatrix} \qquad \text{Eq. 6.1}$$

The same process is applied to the last shoreline measured, this is the 24th November, but the data is de-mean respect to the mean of the initial shoreline surveyed and the same angle of rotation is used.

Similar process is conducted for the cross-shore modelled data. The initial cross-shore coordinates are de-mean to obtain same y-axis units than the measured shoreline. Knowing the location of the groyne in the alongshore direction, it is set at the position (0,0) according to the measured data, then the modelled longshore distance is 250m east to the groyne given in negative values and 165m west to the groyne in the positive direction.

Figure 6.2 represents a comparison between the initial shoreline calculated by the model and the predicted shorelines for the date of the end of the experiment (24th November 2007) for the *k* coefficients obtained at the east and the west sides of the groyne(0.2579 and 0.1508 respectively) considering the shingle fraction and the surveyed grid for longshore sediment transport rate calculations. As expected the shoreline modelled using higher *k* coefficient over-predicts with respect to that one obtained with lower *k* coefficient values. This results in a prediction of approximately 6m more seaward displacement of the updrift shoreline position with respect to the predicted shoreline using k=0.1508.

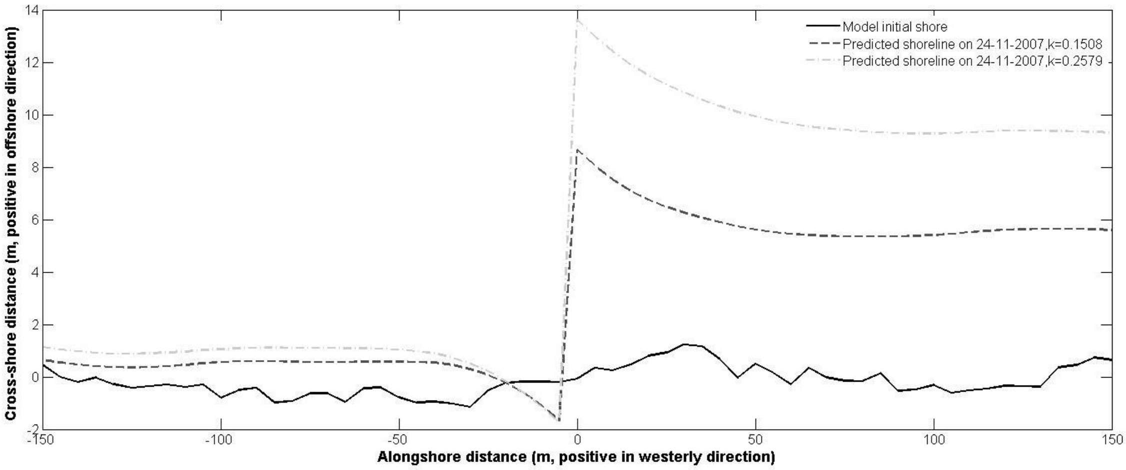


Figure 6.2 Comparison between the initial and final modelled (28th August and 24thNovember) shorelines at contour 1m predicted by the one-line model (total time modelled t=1309) for k=0.1508 and k=0.2579. The groyne is located at the 0m alongshore distance.

Figure 6.3 shows the comparison between the measured shorelines at 1m contour extracted from the topographic beach profile data at the beginning of the experiment period (1st October 2007) and at the end (24th November 2007). It is observed that the effect of the groyne for trapping sediments is demonstrated especially along the first 50m east of the structure, whilst at the same time accretion is also significant along the first 50m west of the groyne. Between the initial and final measured shorelines there is a shift on the cross-shore direction of approximately 2m indicating recession at the downdrift (east of groyne) and accretion at the updrift (west of groyne). Qualitatively the agreement between the measured and the modelled shoreline change can be seen to be encouraging as it presents shoreline advance at the west side and retreat at the east side. However the model over-predicts the shift by 6 to 12 times of that observed

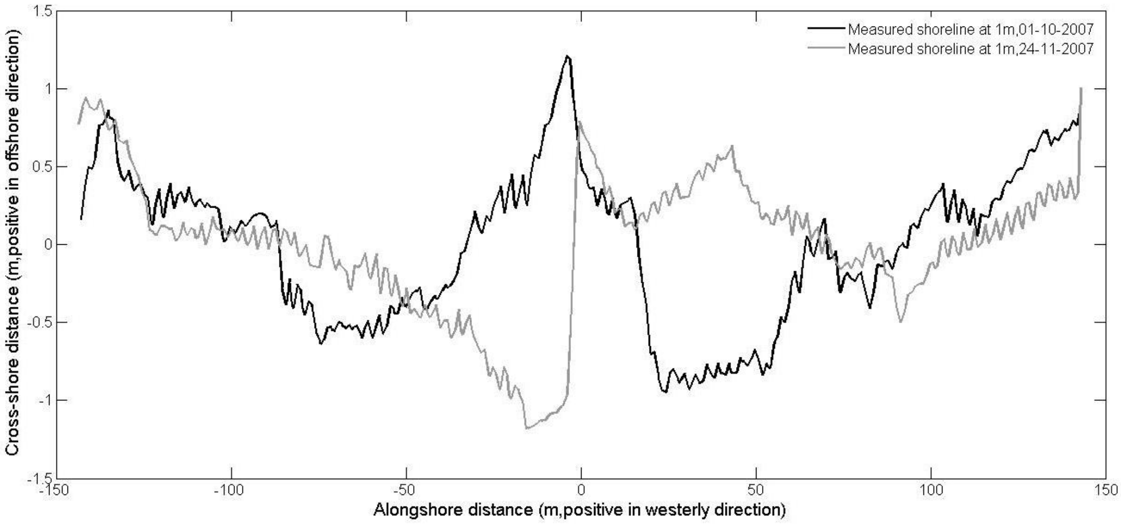


Figure 6.3 Comparison between the initial (1st October 2007, black line) and final (24th November 2007, grey line) measured shorelines at 1m contour extracted from the topographic beach survey data. The groyne is located at the 0m alongshore distance.

It is suggested here that a *k* coefficient of 0.067, closer to that obtained by the conventional analysis, would be more appropriate and provide a better approximation between the modelled and the measured shorelines.

6.3 Discussion and conclusion

The difference between the modelled and predicted shorelines suggest to investigate into the assumptions from the model and those from the analysis presented in this thesis e.g. the wave transformation method, the inclusion of the bathymetry into the analysis and the use of the CERC Equation.

- If the shoreline obtained through analysis of the field measurements of the shingle fraction of the beach (riding over the interface) should be show a similar pattern with that obtained by modelling using a given *k* coefficient, then this might suggest the following:It can be argued that the *k* coefficient chosen is suitable for fitting the one-line model to predict the shoreline evolution. This *k* coefficient is site-specific and for shingle sediment related, it incorporates the influence of the tides by the assumption of determine the 'Active' and 'No active' transport.

- The one-line model is appropriate for predicting the shoreline at this particular site.

Chapter 7

7 Discussion and conclusions

7.1 Discussions

7.1.1 Methodological contributions

7.1.1.1 Effectiveness of the temporary groyne structure

According to different authors, e.g. Brampton and Godberg, 1991; Rogers et al., 2010; Van Wellen et al., 2000 Wang and Kraus, 1999, the groyne structure as an impoundment technique represents an efficient method to calculate LST rates among the existing field methods.

The main characteristic of the groyne considered was its capability for trapping the sediments assuming that is not permeable to the sediments for passing through the structure although the presence of groynes has effects on the hydrodynamic conditions e.g. depending on the groyne design and the materials used for the construction (Reeve et al., 2004; Rogers et al., 2010).

The analysis of the beach profile data indicate that this technique has worked successfully and can be applied for demonstration to the coastal authorities to be a good method to evaluate morphological changes quantifying LST rates related to determined wave and tide conditions in a mixed beach, as Wang and Kraus (1999) concluded from their work applying a short term impoundment technique on a sandy beach.

7.1.1.2 Field data set for a mixed beach

A unique beach profile data set has been produced consisting 32 beach profile lines set out in a regular survey grid on a daily basis over approximately two months using RTK-

GPS. Contemporary measurements of tidal data and wave data have also been collected over the same period.

7.1.1.3 Interface sand- shingle for further investigation

The consideration of the approach interface sand-shingle was adopted in order to investigate the contribution of the coarse sediments as a coastal defence mechanism, and to study the behaviour of the different types of sediment within a mixed beach and their contribution to the longshore sediment transport rates. It is noted that this approach should be consistent in regards with the criteria considered to establish the seaward limit of the boundary between the shingle and the sand. This can affect significantly the beach volume computations to ensure that the range of beach profile change is considered due to longshore sediment transport processes as it is assumed.

7.1.1.4 Estimation of the wave parameters at breaking

The calculation of the wave propagation to the breaker line using the field data applying the Airy Theory may imply some inaccuracies when the different assumptions considered are not consistent, e.g. when estimating significant wave height at breaking as considering an empirical formulation derived for wave height in deep water (Munk, 1949) to be applied to the significant wave data measured by the instrument in intermediate waters. Other existing relationships could be investigated as those obtained by Komar and Gaughan (1972) or Kaminsky and Kraus (1993) (in Komar, 1998). Thus, this possible oversight to consider this approach to estimate Hsb has been introduced to the estimation of the water depth at breaking using the breaking criterion derived by Munk (1949).

7.1.2 Mixed beach morphodynamics

It was demonstrated that the beach morphology responded to the hydrodynamic forces and it was observed by comparing the beach volume distribution with the alongshore wave power. Therefore, the Energy Flux method applied is considered a good approximation to estimate longshore sediment transport rates.

There are some cases when the shoreline changes observed could be due to the shoreline retreat or shoreline steepening. It is thought that the current analysis of the mixed beach profiles related to the different water levels (conventional analysis) could be compared with the analysis of the shingle fraction and investigate whether there is a difference or not in their morphodynamic response.

Cusps are often present at Milford-on-Sea and they were included in the analysis, however, this should be investigated further whether they may have a significant influence in the sediment budgets. This could be one of the reasons why there is a difference of longshore transport rates when calculating the beach volumes considering the surveyed grid or as a unique beach cell.

It is noted an apparent contribution of the swash zone, particularly on steep gravel beaches. Then it is thought that the anomalies in the *k* coefficients may due to the lack of full consideration of the range of the swash zone. This should be taken into account on the cross-shore transport processes and more importantly, to the total longshore sediment transport budget (Bodge and Dean 1987; Van Wellen, 1999).

The analysis suggests further discussion to define the seaward limit of mixed beaches, whether to consider the beach shingle-sand as a unique system, or if the shingle and the sand fractions behave as two distinct systems considering that both move at different and separate transport systems (Kirk, 1980; Saini, 2009).

There were taken into consideration the influence of the tidal level, however the effects of the run up were not considered and the upper limit of the run could be higher that the tide elevation (Reeve et al. 2004).

7.1.3 Calibration of the CERC Equation for a mixed beach

From the results is it thought that perhaps the wave transformation calculated applying Linear Theory is not adequate given the general considerations, therefore the application of a non linear theory could provide more realistic wave conditions at the shoreline. As an example, the Linear Theory considers a natural approximately rectilinear shoreline with no obstacles (Holthuijsen, 2007) and in this case there is a temporary groyne deployed from the upper part of the berm to the low water range of the intertidal zone.

The use of other longshore sediment transport formulae should be investigated for comparison and calibration purposes, as well to assess the capability of other formulae for a mixed beach.

Sensitivity analysis of the longshore transport coefficient k: according to the literature and as it was predicted by the one line model, different *k* values predicted different shorelines. According to literature, *k* for coarser sediments is obtained one or two orders of magnitude less than *k* coefficient for sandy beaches. Then, higher *k* will over predict the shoreline change.

7.2 Conclusions

In summary, this study has carried out a new methodology using an impoundment technique for calculating longshore sediment transport rates on a mixed, shingle and sand, beach. The work was based on an experimental performance that was applied for the first time in the UK and on a composite mixed beach. The aim of the study was to demonstrate the capability of the methodology for estimating an empirical *k* sediment coefficient for this type of beach considering the Energy Flux method and applying the CERC Equation (SPM, 1984).

As the gravel fraction were considered as the main mechanism to protect the cliff under the action of waves and tides, the analysis of the beach profile surveys measured using a

DGPS were based on that gravel fraction of such cross-shore measurements marked by the interface gravel-sand. Then, this allowed calculate the immersed weight longshore transport rates.

In addition to that, wave parameters and tides were also recorded at the field site. These data were used to calculate the empirical longshore wave energy as well as for feeding the one line model tested afterwards.

7.3 Future research

The avenues of further possible research include:

- Apply the Kamphuis 2002 and the Kamphuis and Readshaw (1978) that relates the surf similarity parameter to *k* (Smith et al., 2009).

- From the conclusions and the results of this study, it is proposed apply the methodology using other longshore transport formulae that include further beach parameters as the sediment size or beach slope (Kamphuis, 1986; 2002) for calibration purposes or even other existing longshore transport formulae for shingle beaches (Chadwick, 1989).

- Calculate the run up to investigate the range of action of the water level as it can extend higher than the mean high water spring and therefore the waves enhance the effects of the tides on the beach cross-shore profile.

- Investigate the grain size distribution at Milford-on-Sea and its variability on the cross-shore and alongshore direction according to the changes of the beach profiles wave climate related.

- Also, the use of other data sets from other mixed- shingle beaches will be useful to fit into the model for validation of the *k* coefficient obtained.

- For continuing a preliminary assessment made by the author as part of a conference paper (Martín- Grandes et al., 2010) to relate the onshore-offshore movement of the longshore sediment bar with the gravel beach profiles and wave conditions, a Canonical Correlation Analysis (CCA) will be apply to the data in collaboration with Horrillo-Caraballo from Swansea University.

Appendix A

A.1 Summary of SMP policy at Milford-on-Sea and Hordle Cliff

Table A.1 Summary of SMP policy for Milford-on-Sea and Hordle Cliff (extracted from SMP2 report, Royal Haskoning UK, Section 6, 2011).

Present and Previous Policy						Proposed SMP2 Policy							
							New proposed policy		Policy reverts back to SMP1		Policy in agreement with latest strategy but no with SMP1		
SMP1			Modified Policy			Management Area		Policy Unit		Policy Plan			
MU*	Location	Policy	Ref*	Location	Policy					To 2025	2026 to 2055	2056 to 2105	Comment
CBY 6	Milford-on-Sea	HTL	S1*	Milford-on-Sea	Beach recharge and maintain defences	MA01	Hurst Spit and Milford	CBY.A.2	Milford Seafront	HTL	MR	MR	Investigate options for developing a continuous beach between Rock Cliff and Hurst Spit, subject to funding.
								CBY.A.3	Rock Cliff	HTL	HTL	HTL	Local realignment controlled by hand points.
								CBY.A.4	Cliff Road	MR	MR	MR	Intent to maintain road and property but with possible future need for further realignment beyond the period of the SMP.
CBY 5	Hordle Cliff to Barton-on-Sea	DN/ Retreat	S1*	Hordle Cliff to Barton-on-Sea	Allow natural evolution	MA02	Barton-on-Sea	CBY.B.1	Hordle to Barton	NAI	NAI	NAI	Allow natural rollback.

A.2 Schematic diagrams of temporary groyne construction

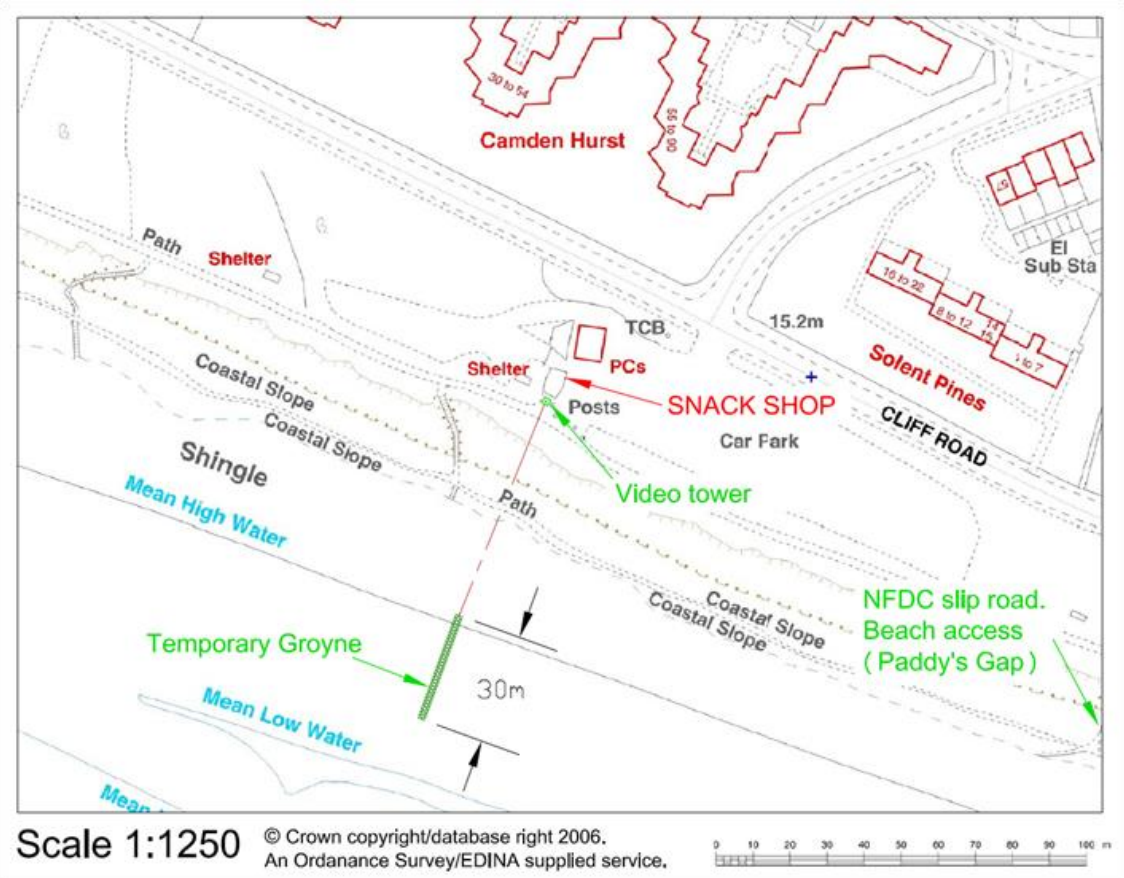


Figure A.1 Plan view diagram of the location of the temporary groyne at Hordle Cliff.

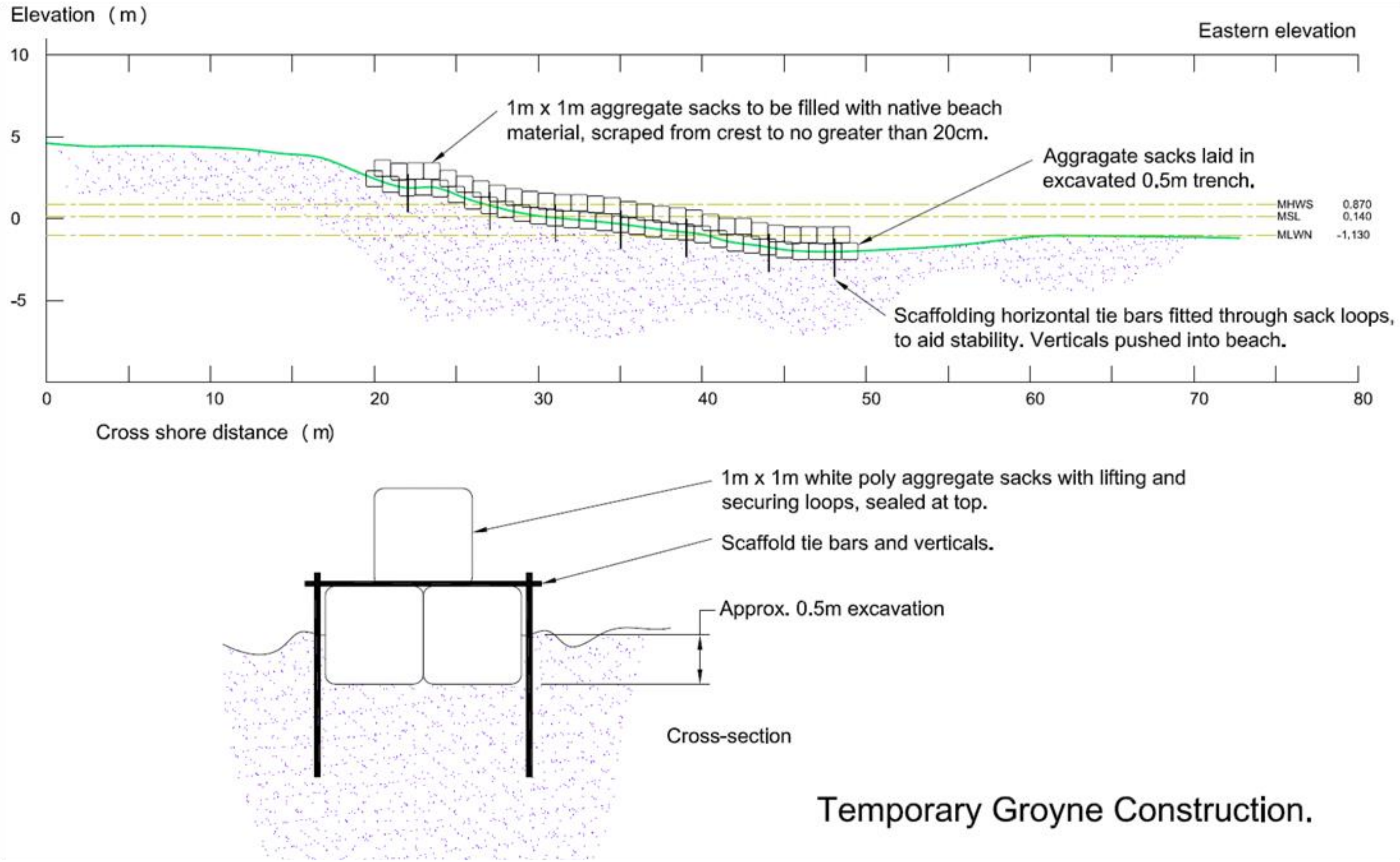


Figure A.2 Top: design of the cross section of the groyne respect to the water levels. Bottom: cross section of the lower part of the groyne.

A.3 AWAC Log configuration settings

```
==============================================================
Deployment   : MilSep
Current time : 28/08/2007 15:27:32
Start at     : 04/09/2007 09:00:00
Comment:
Milford Deployment starting 4th September 07 for 100 days
-------------------------------------------------------------
Profile interval      (s) : 600
Number of cells           : 20
Cell size             (m) : 0.50
Average interval      (s) : 120
Blanking distance     (m) : 0.41
Measurement load      (%) : 62
Power level               : HIGH
Number of wave samples    : 1024
Wave interval         (s) : 3600
Wave sampling rate   (Hz) : 2
Wave cell size        (m) : N/A
Wave AST window start (m) : N/A
Wave AST window size  (m) : N/A
Wave AST find first peak  : N/A
Wave AST qual. threshold  : N/A
Wave AST SUV mode         : DISABLED
Compass upd. rate     (s) : 600
Coordinate System         : ENU
Speed of sound      (m/s) : MEASURED
Salinity            (ppt) : 35
Analog input 1            : NONE
Analog input 2            : NONE
Analog input power out    : DISABLED
File wrapping             : ON
-------------------------------------------------------------
Assumed duration   (days) : 100.0
Battery utilization   (%) : 105.0
Battery level         (V) : 11.9
Recorder size        (MB) : 154
Recorder free space  (MB) : 154.000
Memory required      (MB) : 60.5
Vertical vel. prec (cm/s) : 0.8
Horizon. vel. prec (cm/s) : 2.4
-------------------------------------------------------------
Instrument ID             : WPR 105
Head ID                   : WAV 5070
Firmware version          : 1.17 AST
-------------------------------------------------------------
AWAC AST Version 1.31
Copyright (C) 1997-2007 Nortek AS
==============================================================
```

Figure A.3 AWAC configuration settings at the time of the deployment.

A.4 Grain size statistic formulae

The grain size representative of a sediment sample is given by estimation of its mean, this is usually calculated by the statistical analysis of the length of the intermediate axis (b-axis) of all or a subsample of the individual grains of the sample (Masselink, 2011). The measurement of the b-axis length can be obtained by sieving the sediment samples, once the grain size are known, the size of the sediment sample is represented as the percentage of the weight frequency for the sediment samples analysed, typically approximates a log-normal distribution.

Median diameter M_d and the mean diameter M, define typical sizes of a sample of littoral materials. The median size (mm) is the most common measure of sand size in engineering reports. It may be defined as $M_d = d_{50}$ where d_{50} is the size in mm that divides the sample so that half sample, by weight, has particles coarser than the d_{50} size.

To a good approximation, the median is interchangeable with the mean for most beach sediments. Since the actual size distribution is such that the logarithm of the size is approximately normally distributed, the approximate distribution can be described (in phi units) by the two parameters that describe a normal distribution: the mean and the standard deviation.

- Standard deviation: measure of the degree to which sample spreads out around the mean (i.e. its sorting) using Inman (1952) (Dean and Dalrymple, 2004), σ_ϕ=0 *perfectly sorted sediment*, and σ_ϕ~ *0.5typical well-sorted sediments*, then, if all particles have sizes that are close to the typical size it is said that they are well sorted.

$$\sigma_\varphi = \frac{\varphi_{84} - \varphi_{16}}{2}$$

- Skewness: measures the degree by which the phi-size distribution departs from asymmetry (Inman 1952) (Dean and Dalrymple, 2004), then, $(M_{\varphi} - M_{d_{\varphi}}) = 0$for a perfectly symmetric distribution.

$$\propto_{\varphi} = \frac{M_{\varphi} - M_{d_{\varphi}}}{\sigma_{\varphi}}$$

If the particle sizes is distributed evenly over a wide range of sizes it is said that particles are well graded. The relation between sorting and grading is: well sorted-poorly graded and well graded-poorly sorted.

The skewness and kurtosis indicate how far the actual size distribution of the sample departs from this theoretical lognormal distribution.

- Kurtosis: formula proposed by Inman (1952) (Dean and Dalrymple, 2004).

$$\beta = \frac{(\varphi_{16} - \varphi_{5}) + (\varphi_{95} - \varphi_{84})}{2 * \sigma}$$

Appendix B

B.1 Short term experiment beach profile data coverage at Milford-on-Sea

Daily beach profile measurements, termed GW00 to GW15 and GE00 to GE15 (west and east side respectively) over the 55 days period of the short term impoundment experiment, 1st October – 24th November2007.

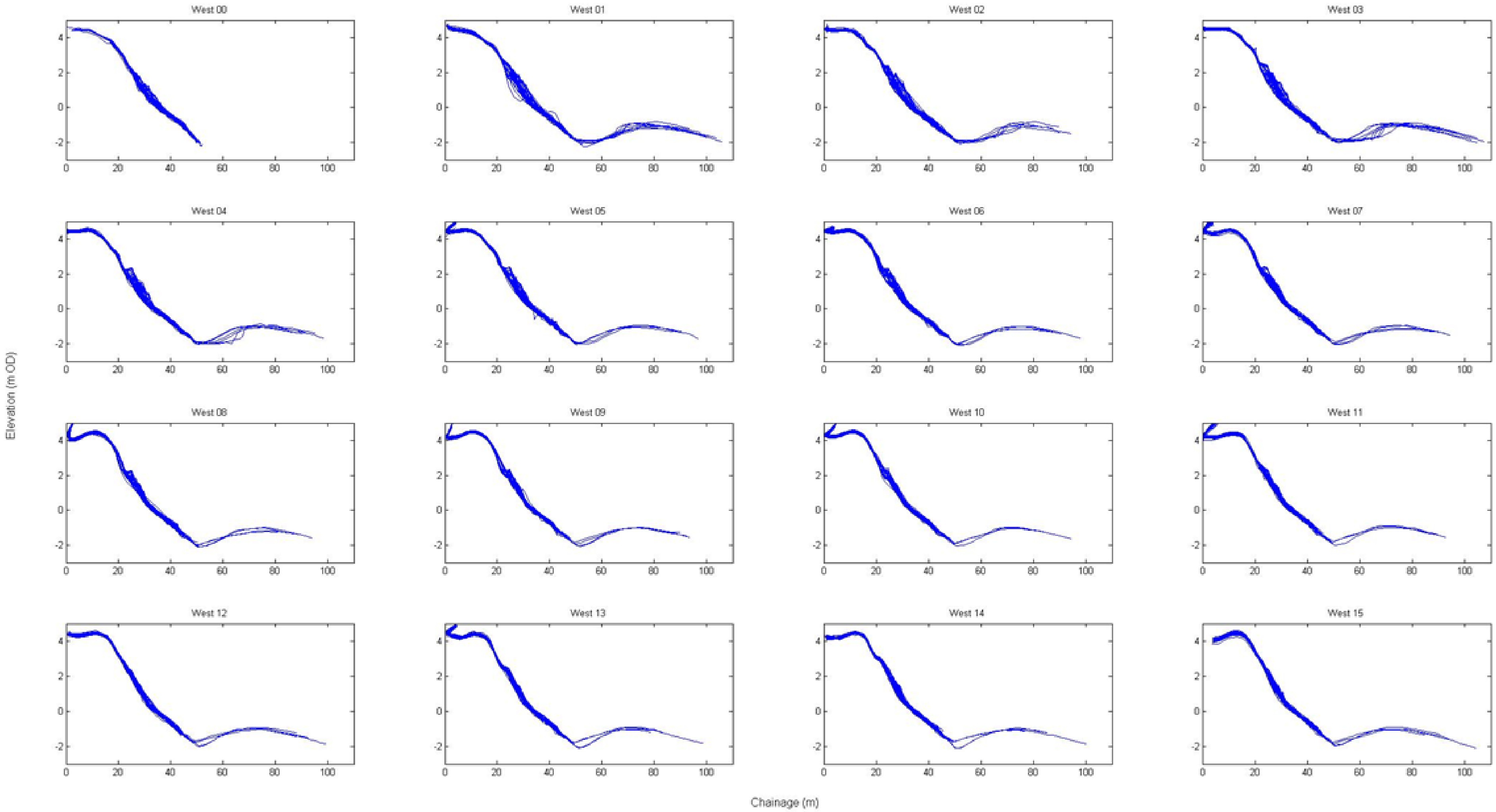


Figure B.1 Time series of the beach profiles measured at the west side of the temporary groyne.

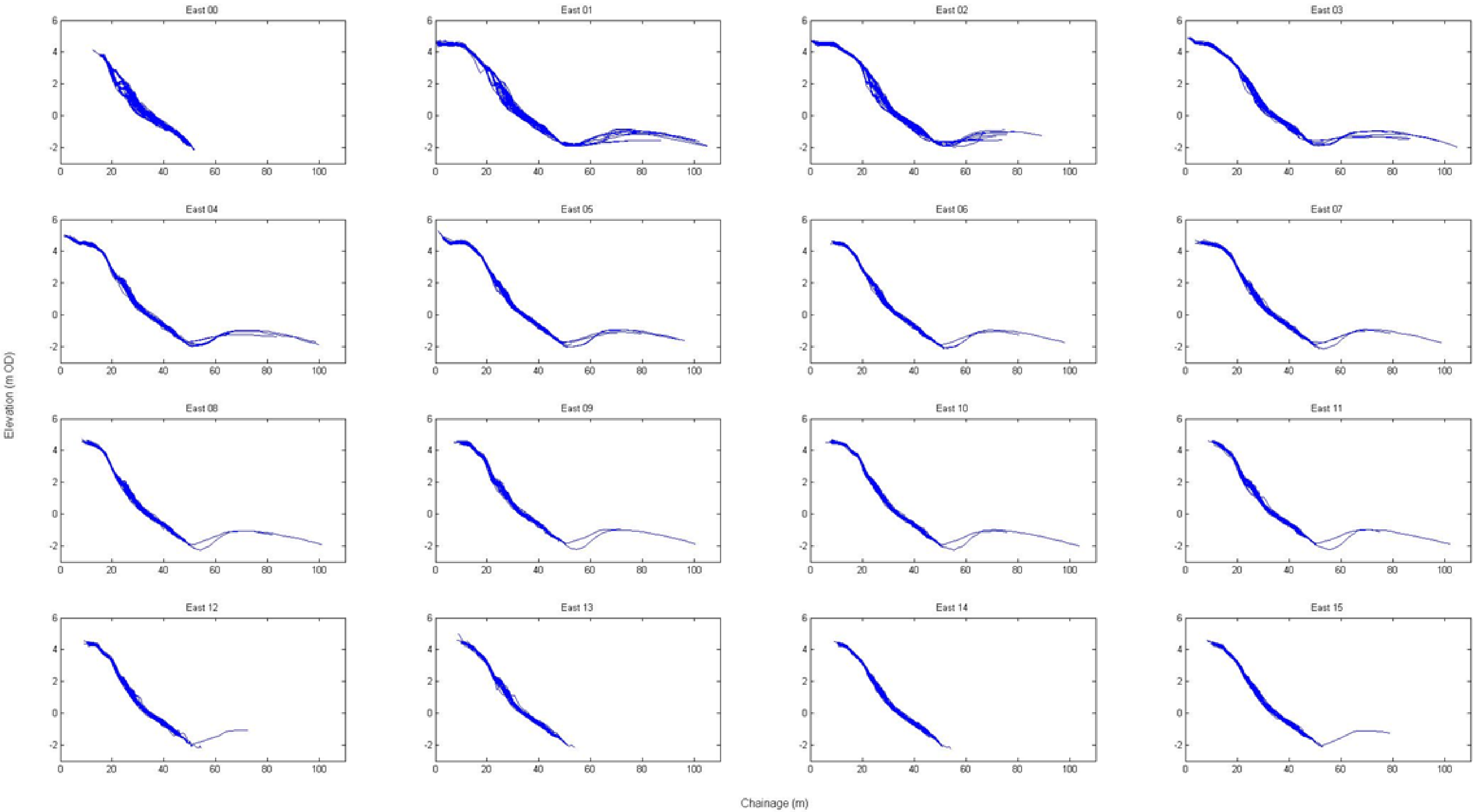


Figure B.2 Time series of the beach profiles measured at the east side of the temporary groyne.

B.2 Measured contour maps and images: Events 1 to 4 for shingle fraction analysis

B.2.1 Event 1: 16th October 2007 storm

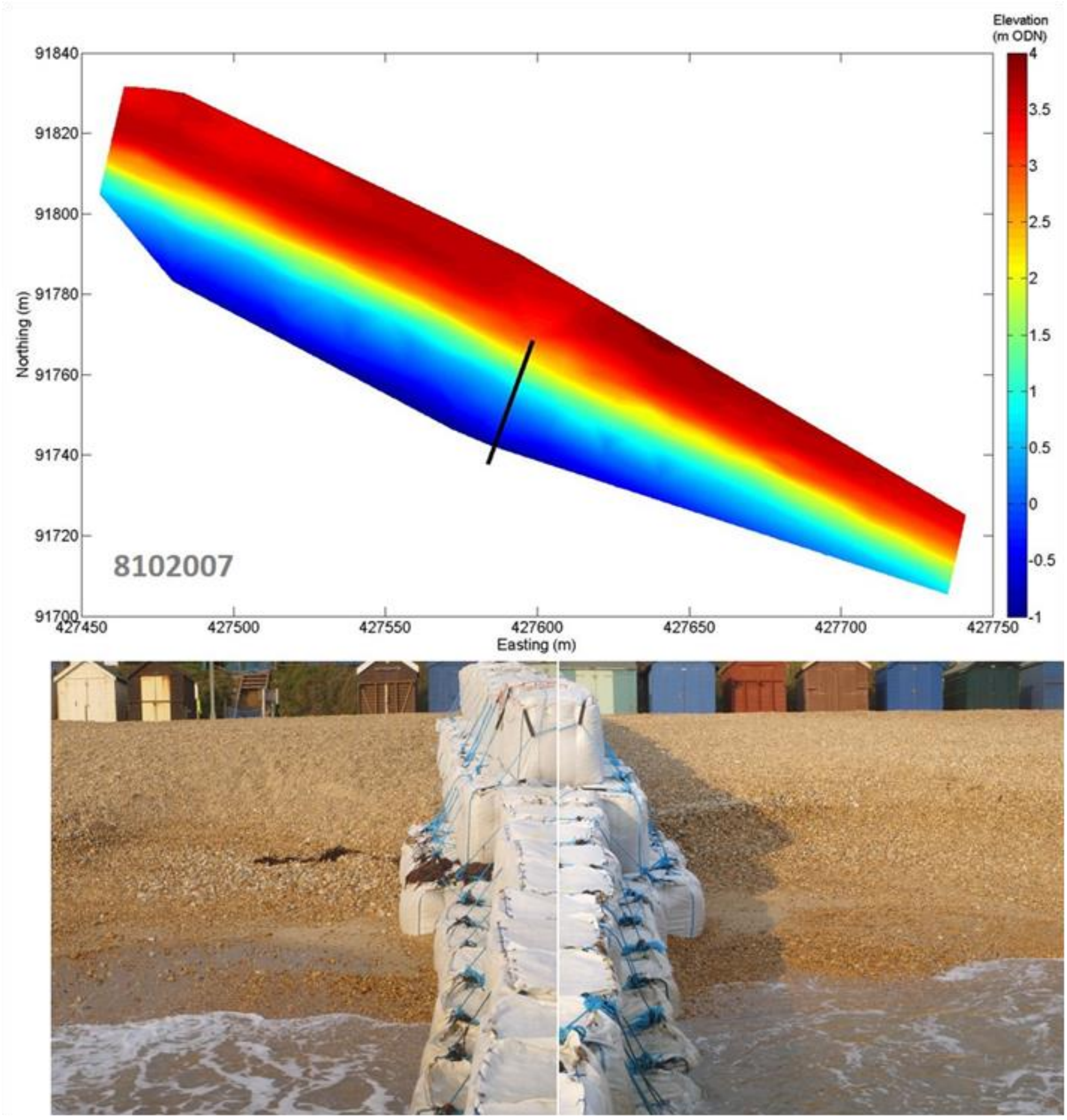


Figure B.3 Top: Contour map of the shingle fraction profiles, the location of the groyne is indicated by the black line; bottom: images taken from the groyne facing landwards (North direction), 8th October 2007. The contour map neither the images show significant beach changes.

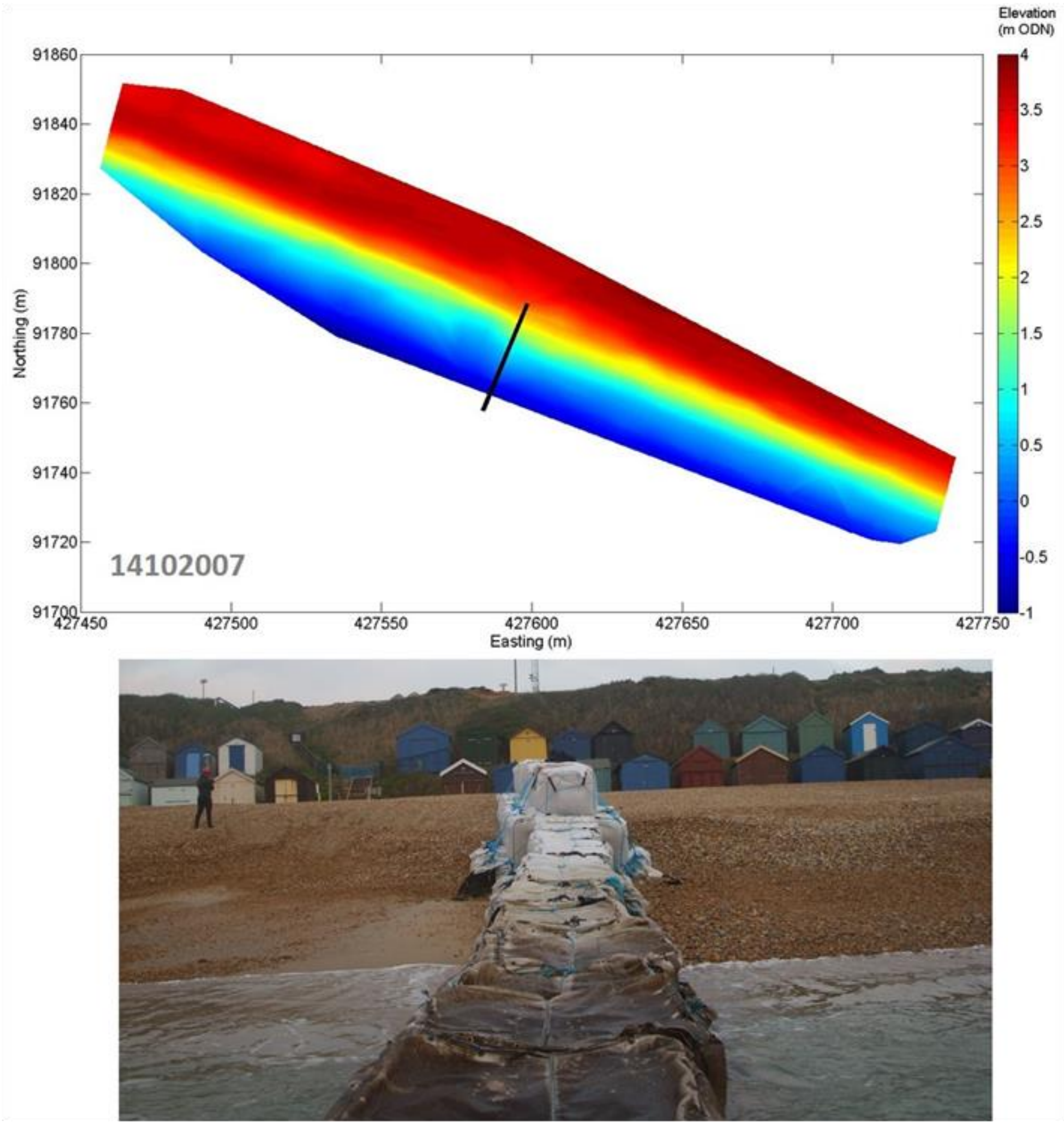


Figure B.4 **Top: Contour map of the shingle fraction profiles, the location of the groyne is indicated by the black line; bottom: image taken from the groyne facing landwards (North direction), 14th October 2007. It is observed accumulation of shingle at the east side of the structure, opposite than the west side. This can explain the recession of the contour between 0.5-1m ODN at the west in this case the downdrift side.**

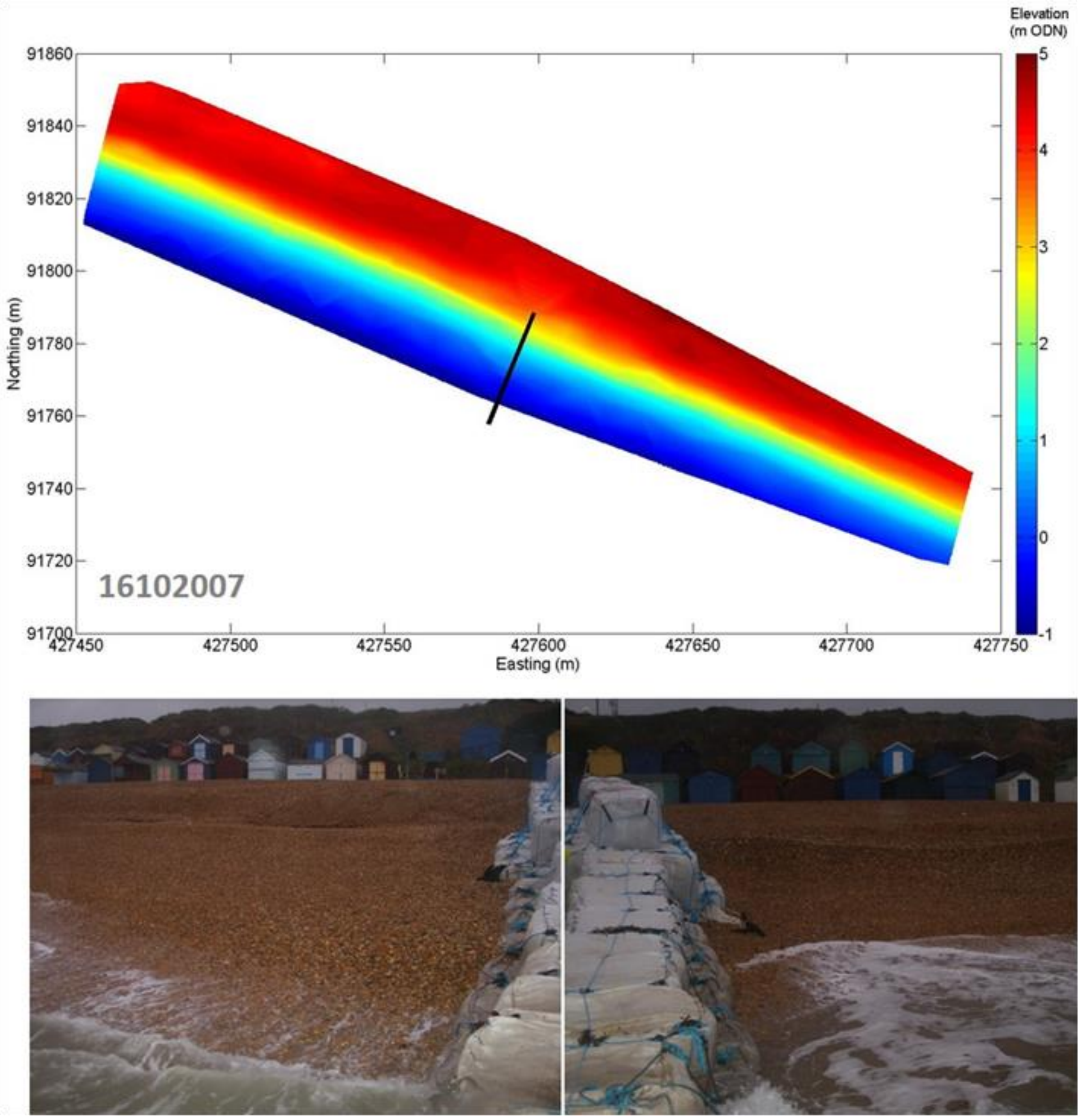


Figure B.5 Top: Contour map of the shingle fraction profiles, the location of the groyne is indicated by the black line; bottom: image taken from the groyne facing landwards (North direction), 16th October 2007. The westerly storm conditions given the 16th caused the accretion updrift (west groyne) observed by the shoreline advance around the 1m contour, and the erosion at the downdrift (east).

B.2.2 Event 2: 28th October 2007 storm

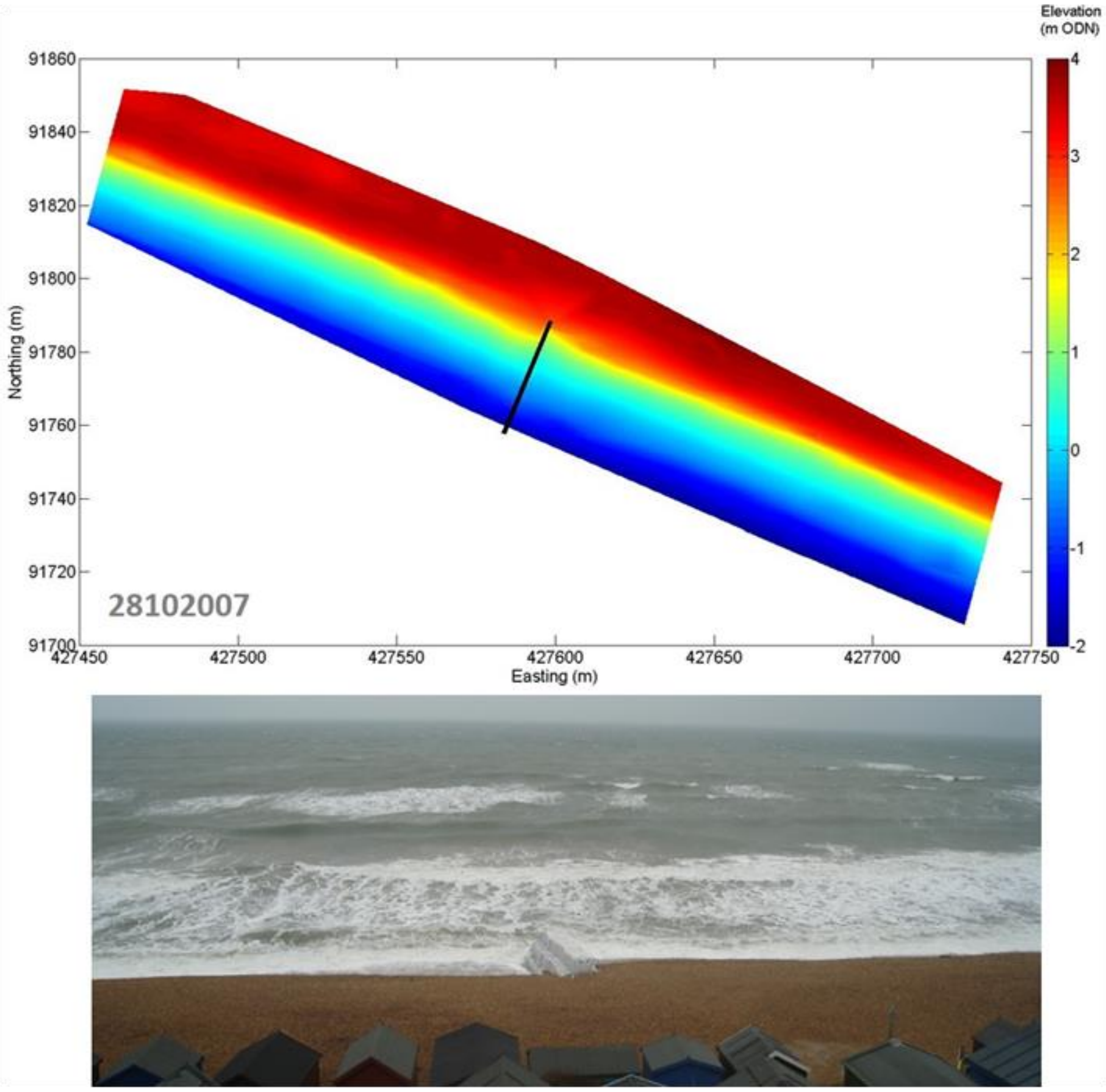


Figure B.6 Top: Contour map of the shingle fraction profiles, the location of the groyne is indicated by the black line; bottom: image taken from the top of the cliff facingSout, 28th October 2007. Thepicture evidence the accumulation updrift and the erosion downdrift observed in the contour map particularly between 1.5 – 2.5m contours.

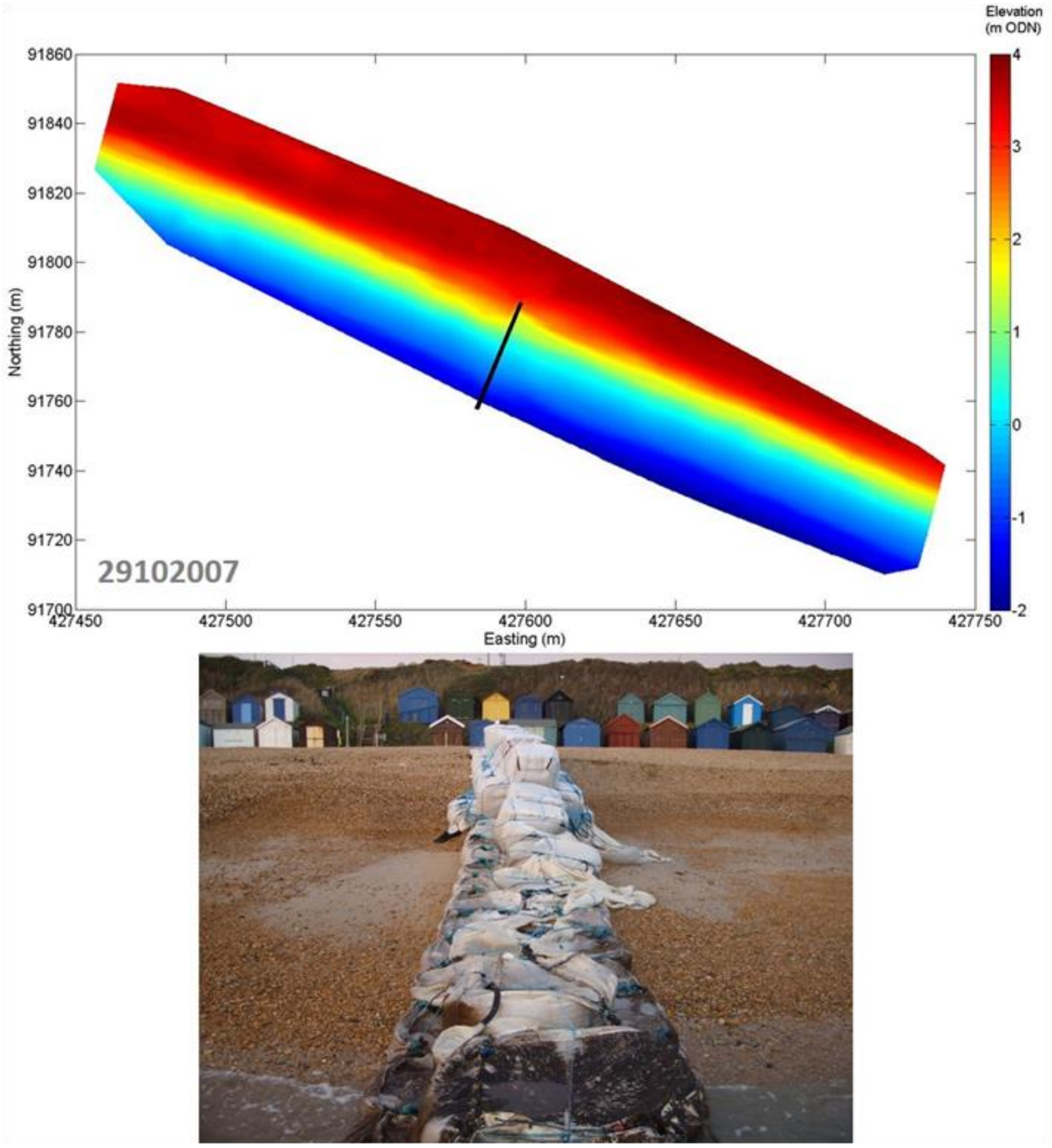


Figure B.7 **Top: Contour map of the shingle fraction profiles, the location of the groyne is indicated by the black line; bottom: image taken from the groyne facing landwards (North direction), 29th October 2007.**

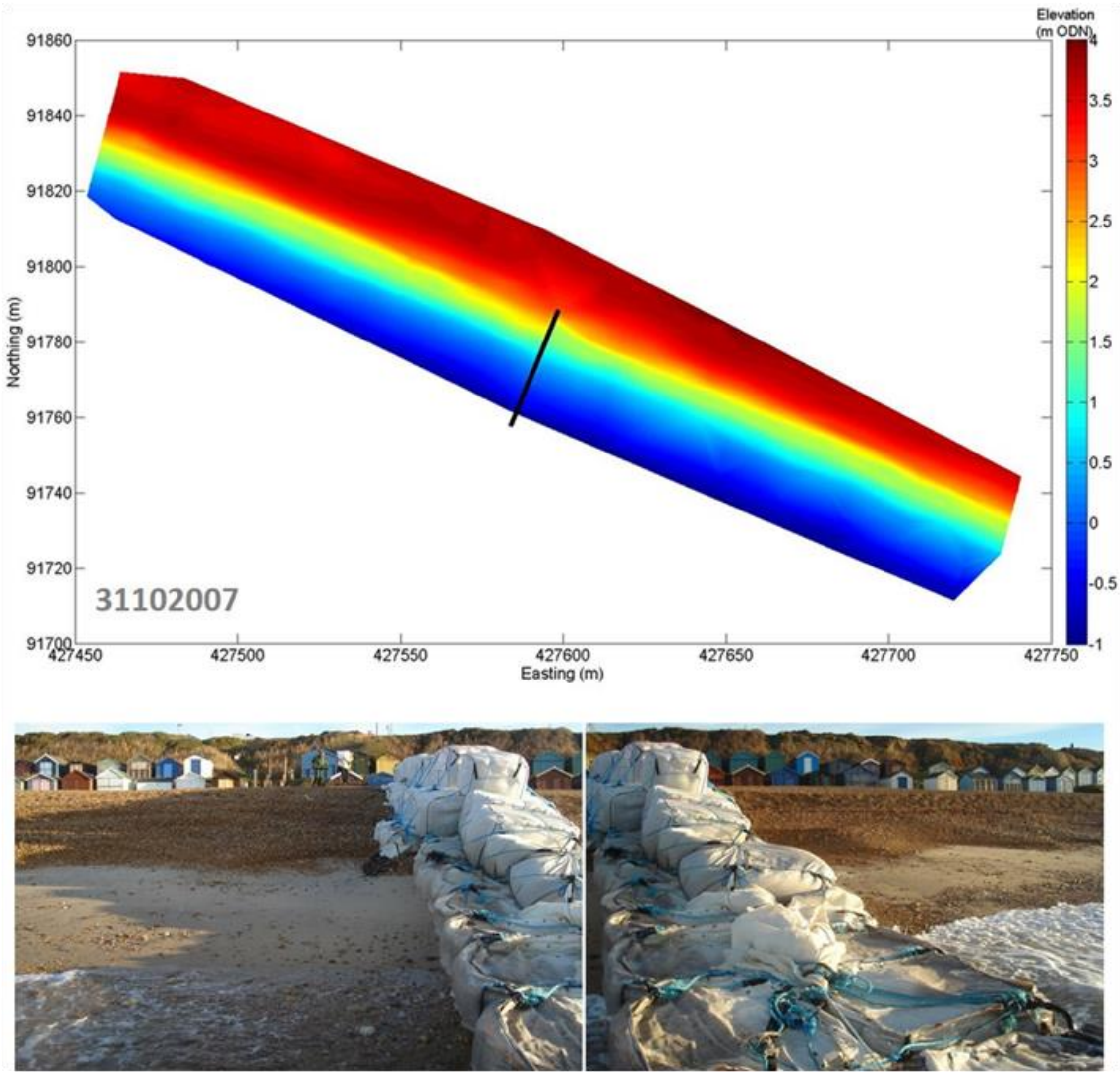


Figure B.8 Top: Contour map of the shingle fraction profiles, the location of the groyne is indicated by the black line; bottom: images taken from the groyne facing landwards (North direction), 31th October 2007.

B.2.3 Event 3: Predominant longshore sediment transport (evidence)

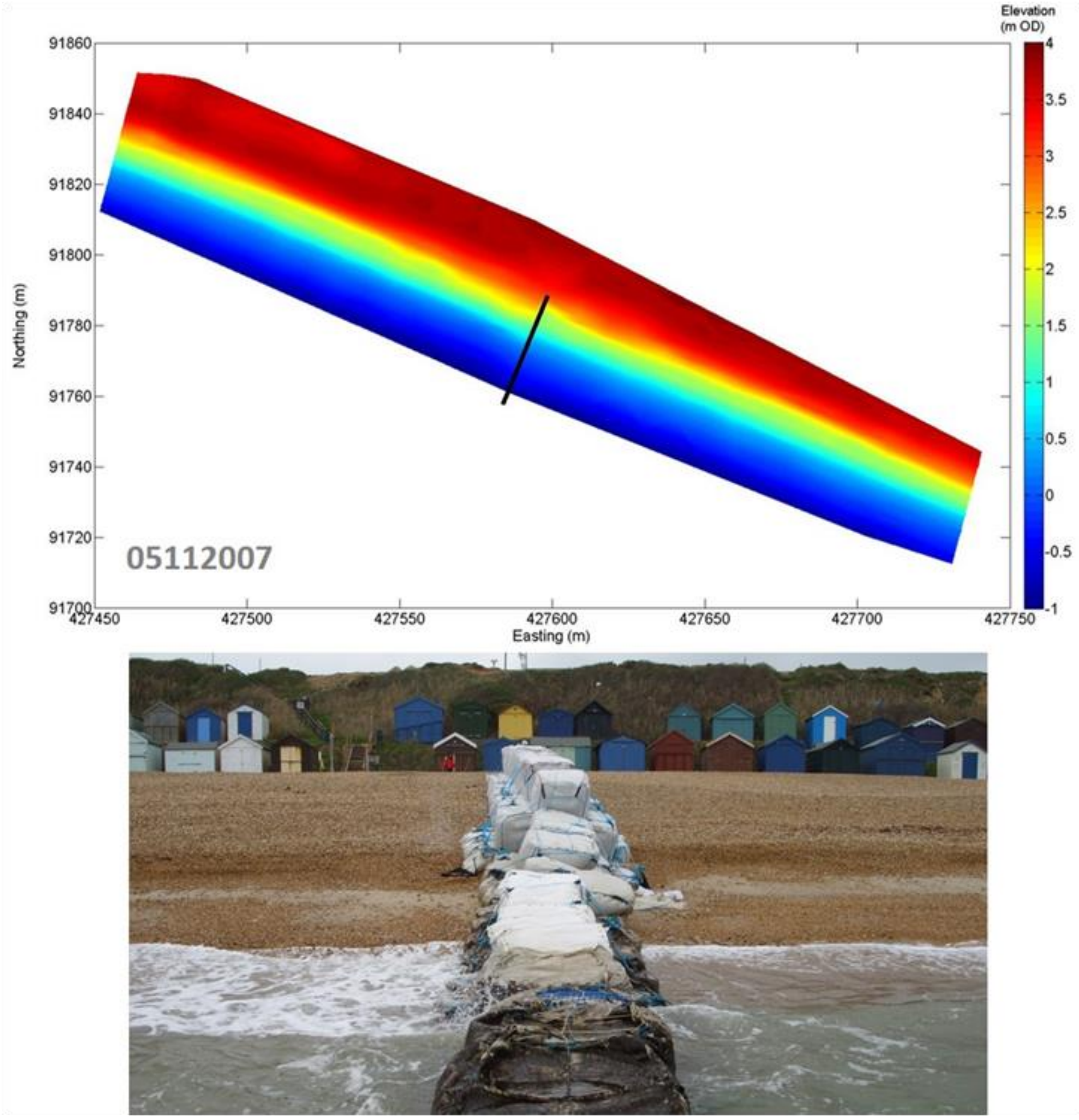


Figure B.9 Top: Contour map of the shingle fraction profiles, the location of the groyne is indicated by the black line; bottom: image taken from the groyne facing landwards (North direction), 5th November 2007.

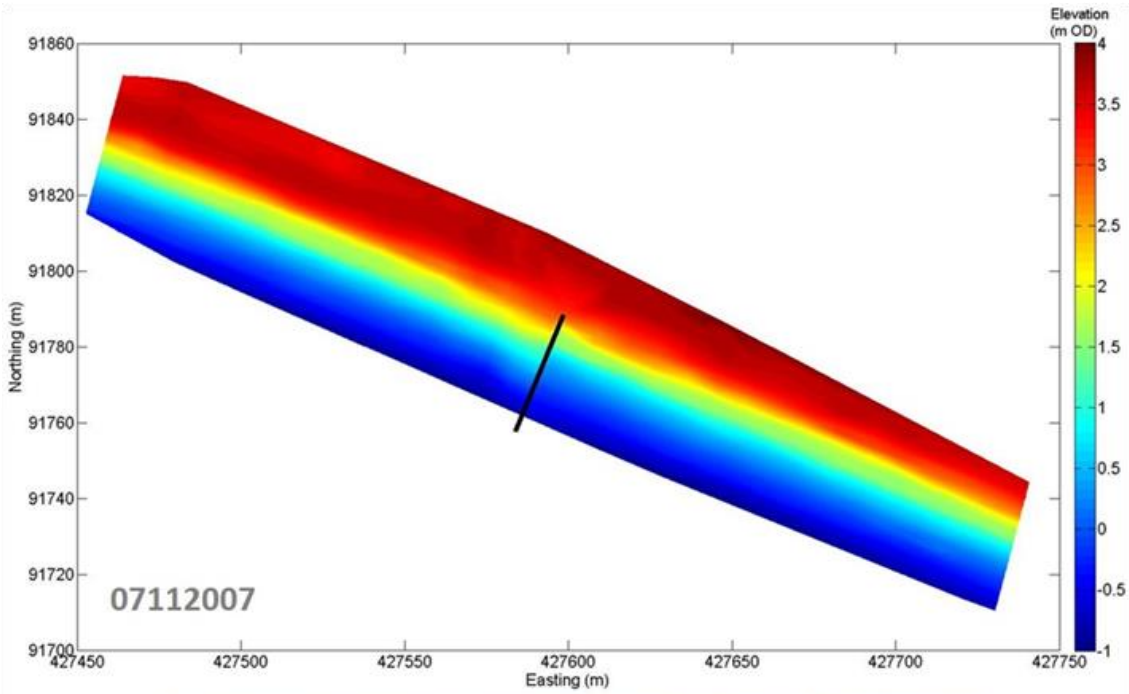


Figure B.10 Top: Contour map of the shingle fraction profiles, the location of the groyne is indicated by the black line; bottom: image taken from the groyne facing landwards (North direction), 7th November 2007.

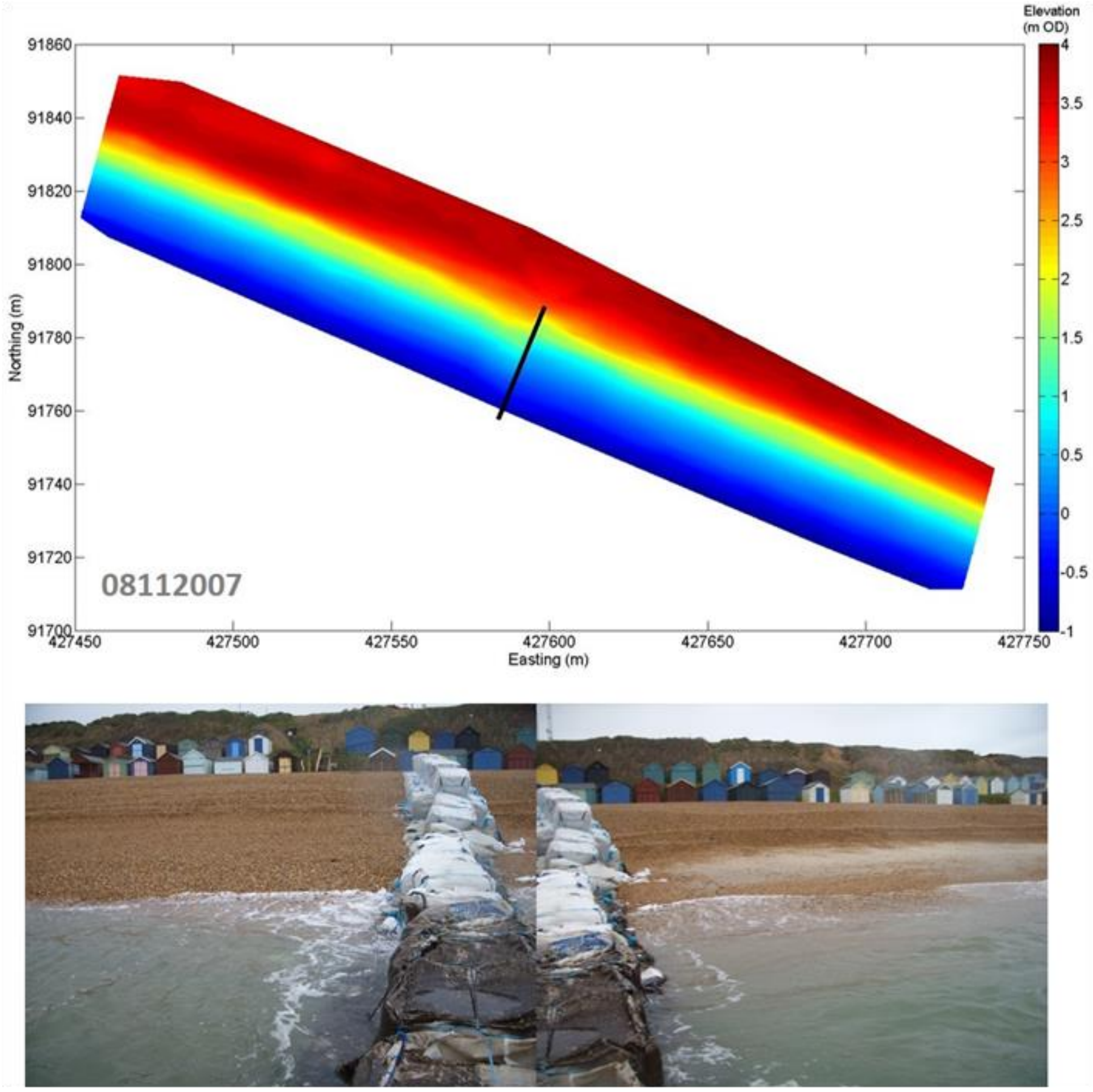


Figure B.11 Top: Contour map of the shingle fraction profiles, the location of the groyne is indicated by the black line; bottom: images taken from the groyne facing landwards (North direction), 8th November 2007.

B.2.4. Event 4: 18th November 2007 storm

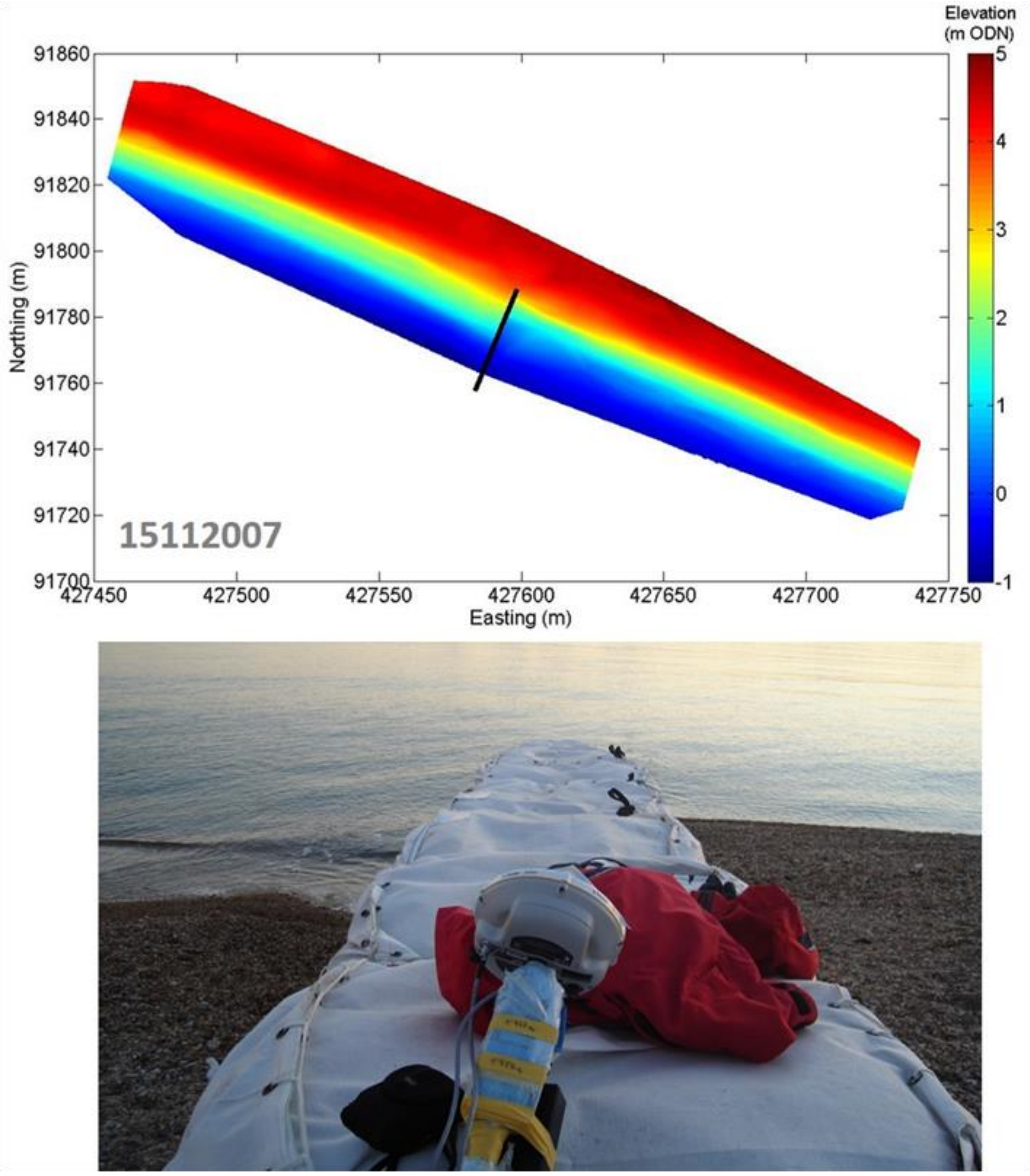


Figure B.12 Top: Contour map of the shingle fraction profiles, the location of the groyne is indicated by the black line; bottom: image taken at the top of the beach facing South, 15th November 2007.

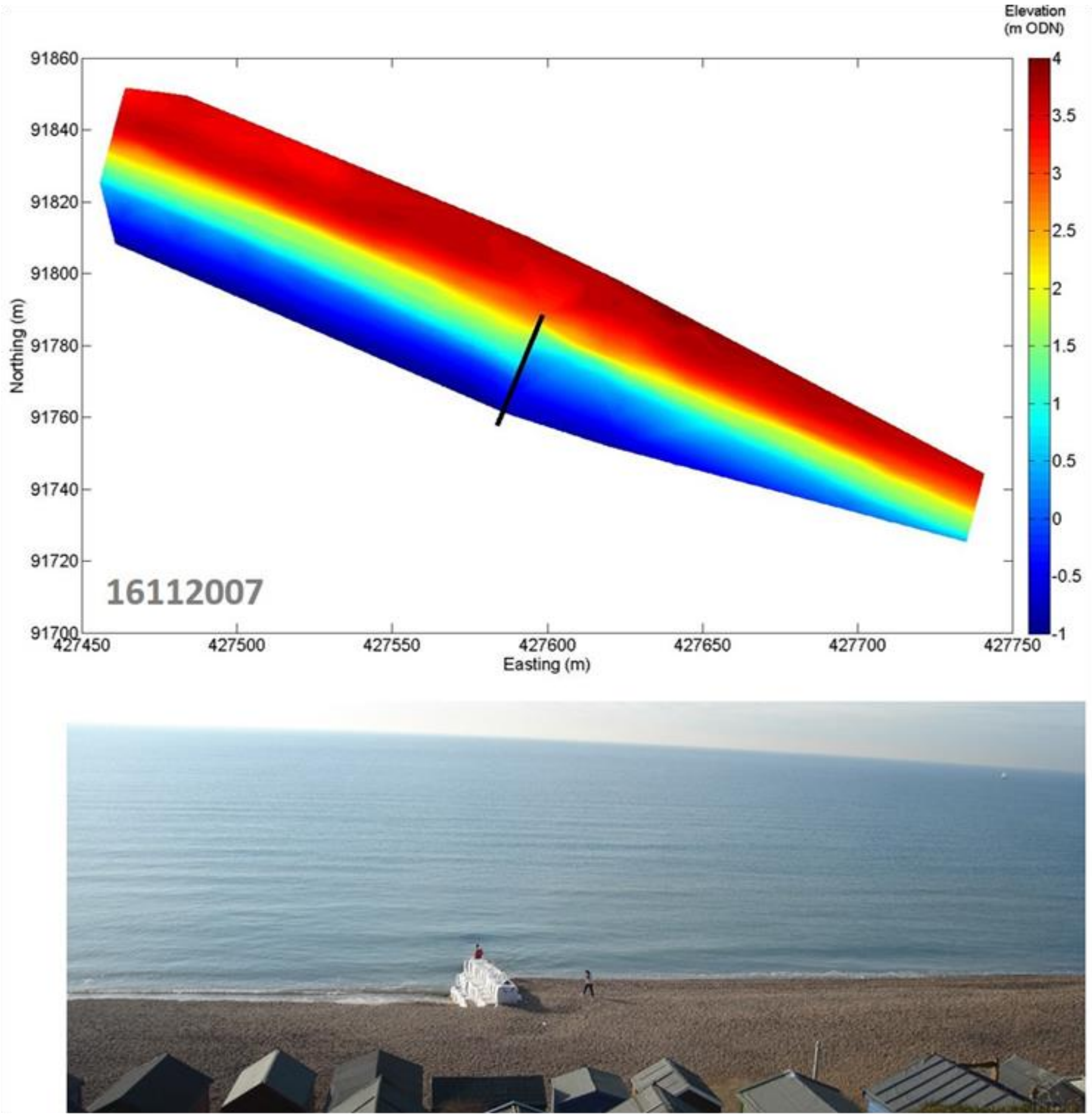


Figure B.13 Top: Contour map of the shingle fraction profiles, the location of the groyne is indicated by the black line; bottom: Image taken from the top of the cliff facing South, 16th November 2007.

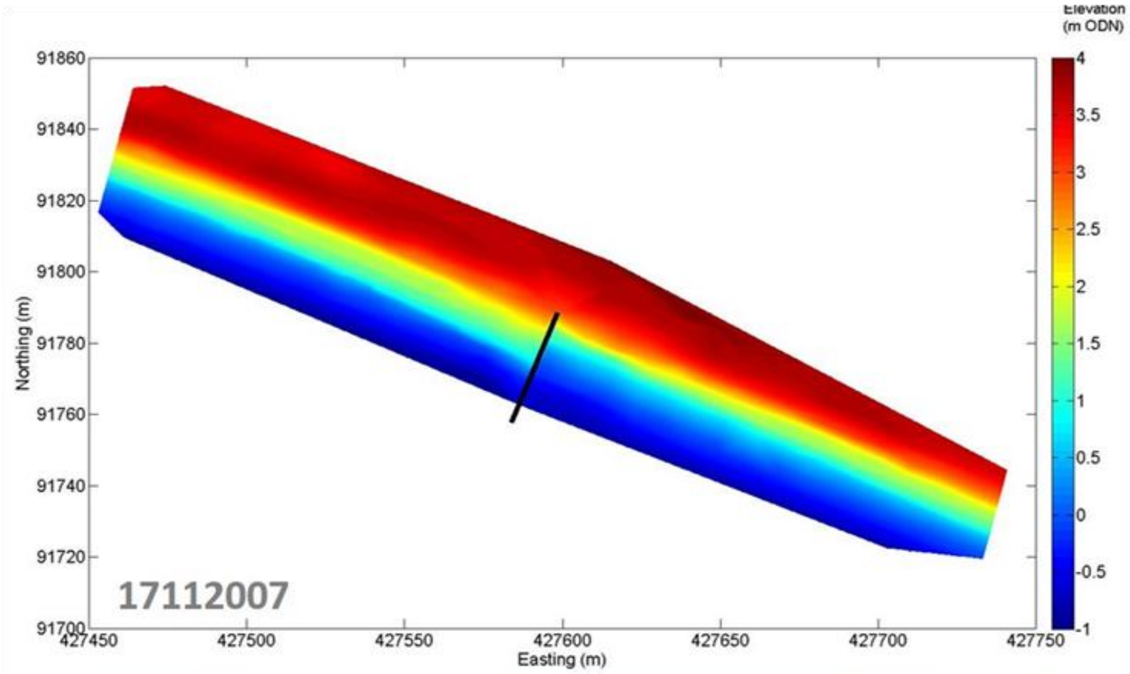


Figure B.14 Top: Contour map of the shingle fraction profiles, the location of the groyne is indicated by the black line; bottom: Image taken from the top of the beach facing South, 17th November 2007.

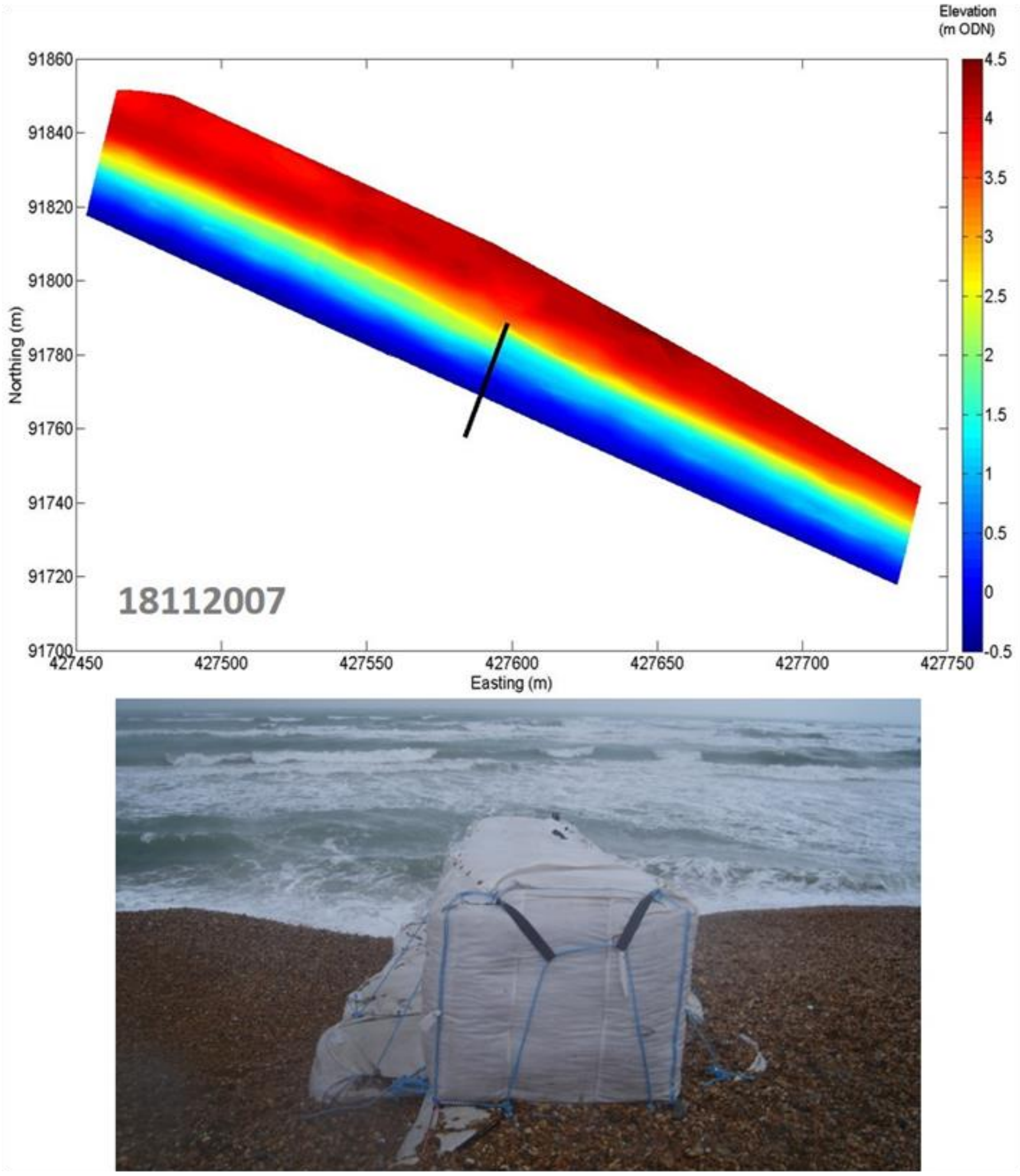


Figure B.15 **Top: Contour map of the shingle fraction profiles the, the location of the groyne is indicated by the black line; bottom: Image taken from the the top of the beach facing South, 18th November 2007.**

List of symbols and abbreviations

Roman symbols

a'	relation of volume solids between the total volume
b	(Subscript), in the breaking zone
c	wave celerity
$c0$	wave celerity in deep waters
D	Sediment size
dx	Longshore distance between contiguous beach profile lines
i	Subscript to indicate the number of profile line
I_l	Immersed longshore transport rate (N/s)
k	Longshore sediment transport coefficient
k_0	Wave number related to the wavelength in deep water
$\overline{k_l}$	Alongshore mean sediment transport coefficient
h	Water depth
H_s	Significant wave height
H_{rms}	Root mean square wave height
l	(Subscript), referring to longshore direction
L	Wavelength
L_0	Wavelength in deep water
lst	(Subscript), referring to longshore sediment transport

m Beach slope

P_{ls} Longshore wave power (N/s)

Q Longshore sediment transport rate

Qe Longshore sediment transport rate at the East side of the groyne

Q_{in} Longshore transport rate transferred into the beach cell

Q_{out} Longshore transport rate transferred out from the beach cell

Qw Longshore sediment transport rate at the West side of the groyne

s (Subscript), related to significant wave

s (Subscript), related to sediments

t Time

T Wave period (also referred as the mean wave period in the analysis, Chapter 4)

T_m Mean wave period

T_p Peak wave period

V Beach volume

y beach elevation variable in the linear polynomial equation (Eq. 4.6)

y y=k0h in Hunt (1979)

z elevation of the daily mean shingle- sand interface

0 (Subscript) related to deep water

Greek symbols

α	wave direction
α_b	wave direction at breaking
ξ_b	surf similarity parameter
η	tide elevation
Φ	phi- scale referred in phi- units
ρ	density of sea water
ρ_s	density of sediments
σ^2	standard deviation

Abbreviations

ABMS	Argus Beach Monitoring System
AWAC	Acoustic Wave and Current
BMAPA	British Marine Aggregates Producers Association
CBY	Christchurch Bay
CCO	Coastal Channel Observatory
CERC	Coastal Engineering Research Center
CIRIA	Construction Industry Research and Information Association
DGPS	Differential Global Positioning System
EA	Environmental Agency

GE	Groyne East: referring to beach profile lines at the east side of the groyne
GL	Groyne Line: beach profile line named GL.
GW	Groyne West: referring to beach profile lines at the west side of the groyne
LST	Longshore Sediment Transport
MHWS	Mean High Water Spring
MLWS	Mean Low Water Spring
MWL	Mean Water Level
ODN	Ordnance Survey Newlyn
PCO	Plymouth Coastal Observatory
RBR TWR-2050	RBR Tide and Wave Recorder 2050, RBR Ltd.
RF-PeBLE	Risk- based Framework for Predicting Long- term Beach Evolution
RTK-GPS	Real Time Kinematic Global Positioning System
SCOPAC	Standing Conference on Problems Associated with the Coastline
SMPs	Shoreline Management Plans
SPM	Shore Protection Manual
TCE	The Crown Estate
USACE	United States Army Corps of Engineers

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THE BAR-BERM DYNAMICS OF A COMPOSITE BEACH

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INTRODUCTION

Gravel and mixed (sand and shingle) beaches are distinctive coastal features along the south coast of the UK and constitute the best natural mechanism of defence against shore erosion and flooding. Despite being of considerable interest to coastal authorities, to date there have been few investigations related to coarse sized sediments (Van Wellen et al., 2000). The harsh conditions of these coastal environments confine the deployment of delicate instrumentation for data recording and as a result there is a scarcity of measurements for gravel and mixed beaches to predict their morphodynamic behaviour over long time scales (Van Wellen et al., 2000; Bradbury et al., 2003).

The purpose of this work is to investigate the correlation between a longshore intertidal sandbar and the berm beach, on a composite beach, using a video-based Argus Beach Monitoring System (ABMS) and field data recorded at the field site.

FIELD SITE

The site of the study is Milford-on-Sea, South UK. The beach is located in Christchurch Bay which forms part of a managed coastal cell. The beach presents gravel size sediments above mean sea water level (MSWL), sand size sediments below MSWL and a sandy longshore bar exposed during spring tides. Cusps are formed under normal wave incidence and low wave energy conditions. The position of the interface gravel-sand has a linear relation with the position on the beach elevation and on the cross-shore distance giving a foreshore beach slope within the range 1:6 and 1:8, typical of mixed and gravel beaches (Van Wellen et al., 2000). Thus, at this site it is assumed that the shingle upper beach determines the performance of the beach as a defence. During storm wave conditions up to three bars can be seen and it is the control that they exert on the beach that is the focus of this work.

METHODOLOGY

The Argus Beach Monitoring System (ABMS) was set at the top of Hordle Cliff. This site has five video cameras imaging several kilometres of beach. Using image rectification and analyses of pixel intensity, it is possible to track beach morphological change and bar movement along the coast. Data collected during an intensive two-month period of measurements and intensive monthly surveys carried out over two years is being used to examine the relationship between intertidal bar position and the beach movement (see Figure 1).

Considering a video-technique based on wave dissipation, the intensity maxima from the time-averaged video images of the nearshore zone have indicated a sensible representative for sandbar location (Kingston et al., 2000).

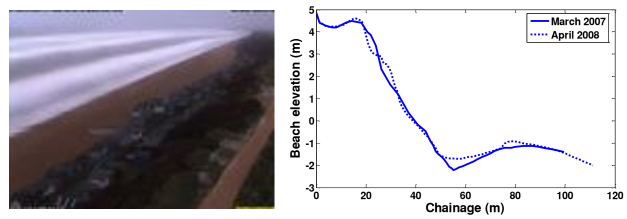

Figure 1 - Left hand side: Timex image from ARGUS cameras c1from field site, Hordle Cliff during the storm of 10th November 2008. Right hand side: cross-shore beach profile showing intertidal longshore bar surveyed for March 2007 and April 2008.

With this methodology we will demonstrate whether there is a negative feedback operating between the beach and the alongshore intertidal sandbar that acts as protection for the shingle berm beach.

This has more than scientific interest because the existence (or lack of it) of a relationship bar-berm will have implications for the management of this beach, including current policies governing dredging and dumping beach materials in the nearshore zone.

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Paper No.

NOVEL METHODOLOGY FOR ONE LINE MODEL CALIBRATION USING IMPOUNDMENT ON MIXED BEACH

Inés Martín-Grandes[1], Jason Hughes[2], David J. Simmonds[3], Andrew J. Chadwick[4] and Dominic E. Reeve[5]

Abstract

Mixed beaches are generally rare worldwide however they are common coastal features along the south coast in the UK and constitute the best natural defence to protect littoral environments against flooding and coastal erosion. To date, only a few investigations have attempted to estimate the LST rate for coarse grain size beaches, using models derived from the CERC equation giving K values of about 8% of those obtained for sandy beaches. In this work an impoundment technique has been applied for the first time on a mixed beach to calibrate the CERC equation using field data. Volumetric estimations from beach profile surveys and wave measurements recorded at the field site, Milford-on-Sea (South UK), are being used to estimate the LST rate, thus enabling model calibration for future shoreline change scenarios.

Key words: Mixed beach, calibration, LST rate, CERC formula, One-Line model.

1. Introduction

The growth of communities in littoral boundaries and the action of coastal processes raise the vulnerability to flooding and erosion. Coastal environments are natural systems susceptible to change under the action of waves and tides and human intervention. Nowadays local authorities request more information about shoreline changes or beach morphodynamics to integrate sustainable activities within the environment under the framework of coastal management plans. Thus, further research in coastal engineering and management focusses on providing guidelines for the best practice to protect littoral environments.

Beaches provide a natural defence against shore erosion and flooding and their configuration is the result of the balance between the action of waves and tides and the sediments moving along the coast. Mixed (shingle and sand) and gravel beaches are common features along the coast in the South of the UK, and have great significance for the protection of coastal communities and environmental and agricultural resources (Mason et al., 1997; Mason and Coates, 2001). However, despite being of considerable interest to local authorities, mixed and shingle beaches are less well understood than sandy beaches (López de San Román-Blanco, 2003). Examples of research into mixed beaches in terms of their dynamics and predictive tools for their behaviour are few and far between: Kirk, 1980; Mason et al., 1997; Mason et al., 2001; Bradbury et al., 2003; López de San Román-Blanco, 2003; López de San Román-Blanco et al., 2006; Buscombe et al., 2006. In these, there is much mention of the need for further research, including the calibration of existing methods derived from laboratory experiments or physical models.

The main factor in the long term development of a beach is the longshore component of the sediment transport along the coastline due to the interaction of waves and tidal currents (Simm et al., 1997). Quantifying the longshore sediment transport, LST, constitutes an essential piece of information for beach management in coastal engineering. Because spatial and temporal changes in LST along the coastline are inevitably linked to beach profile changes over both short and long term (Horikawa, 1988), measurements

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of beach profiles are used to estimate variability in relation to meteorological forcing and to monitor the changes of beach volume and the shoreline position. Thus, the shoreline position, defined by Komar (1976) as "line of demarcation between the water and the exposed beach" is defined parametrically in various ways (Farris and List, 2007).

One of the most familiar formulae for LST in coastal engineering is the CERC equation (SPM 1984) which has been developed for sandy beaches. This empirical equation is based on the Energy Flux method which considers that the immersed weight of the alongshore moving sediment is proportional to the alongshore wave power per unit length of beach (Kamphuis et al. 1986). This equation, specially the one-line model approach, is explained in more detail in the section 4.1 of this paper. A field experiment based on an impoundment technique has been executed in this study to attempt to calibrate the CERC Equation for a mixed beach. The technique consisted on a temporary groyne deployed at the field site as a barrier to the sediments moving alongshore in order to estimate LST rate for this specific site.

In the work presented here, a one-line modelling framework coupled with a wave computation model (Li, 1994) is used to predict the shoreline response to the groyne being placed at the site. This approach allows the beach-wave interaction to be captured in the simulations. The model of the shoreline movement includes both cross-shore and longshore sediment transport. The method of lines with an adaptive time step technique is used to solve this equation. The wave model is driven by offshore wave data, and is solved to provide the wave height, bearing and water depth at the breaking line. This information is then used to calculate the sediment transport rate variation along the section of coastline, using the CERC formula. Results will be presented comparing the model output and the observed shoreline response from field surveys.

2. Field Site

The study area is Milford-on-Sea, located in Christchurch Bay (Hampshire, UK), see Fig. 1. The interest of this area lies on the typical coastal elements presented: cliff eroding at the western side near Barton on Sea; natural beach at Hordle Cliff; coastal defence structures comprising timber groynes and seawall between Milford on Sea and Hurst Spit.

The field site is subject to predominant SSW wave direction (see Fig. 1) and semi-diurnal tides with a spring tidal range of 2m OD.

Figure1.Left-hand panel: Field Site location, South UK. Right-hand panel, Milford on Sea view from Hordle Cliff, bar exposed during spring tide.

A sediment transport study (SCOPAC, 2003) reveals the relatively recent geological origin of Christchurch Bay. Its configuration has been formed by the coastline retreat during the mid to late Holocene transgression in the Quaternary Period.

The coastline from Barton on Sea to Hurst Spit is designated a Site of Special Scientific Interest (SSSI). Milford on Sea is a Coastal Area of Outstanding Natural Beauty (AONB) (SCOPAC, 2003). In terms of coastal planning, Milford is subject to New Forest District Council and is covered by the Solent Strategic Guidance Plans and Local Authority Coastal Management Plans.

2.1. *Mixed Beach*

Mixed beaches are complex systems where the hydrodynamic processes have various effects due to the mixture of sediments and their hydraulic properties (Kirk, 1980; Mason et al. 1997). Hordle Cliff is a mixed (shingle and sand) beach dominated by coarse size sediment acting as the main mechanism to protect the cliff.

Milford-on-Sea shows gravel size above high water level, mixed sediment in inter-tidal beach, sand size below low water line, a longshore bar exposed during spring tides and cusps formed under low energy and shore normal wave incidence, see Fig.2. According to these characteristics, Hordle Cliff may be identified as a composite mixed beach (Mason et al., 1997) that is analogous to the "composite gravel beach" according to the morphodynamic model proposed by Jennings et al. (2002) based on the study of 42 gravel beaches of the South Island, New Zealand.

Figure 2.Characteristics of field site, Milford-on-Sea. On the left-hand panel, sandy bar exposed during spring tide; centre, mixed (shingle and sand) sediments; right-hand panel, coarse grain size on the foreshore slope over a low terrace of sand looking eastwards.

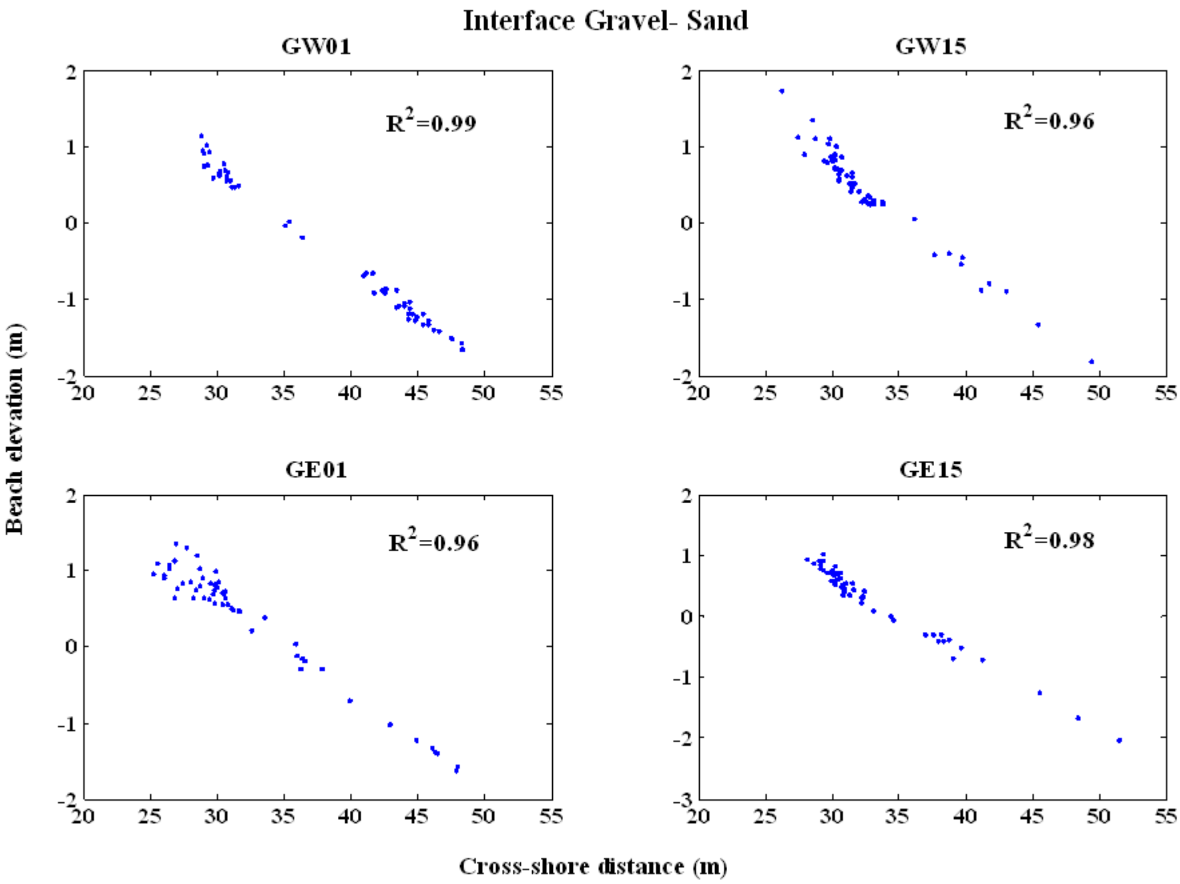

Figure3.Location of the gravel-sand interface along the cross-shore profile for representative profile lines at each side of the groyne (GW, western side and GE eastern side of the groyne).

We may classify Milford on Sea as a composite beach after having analysed the movement of the gravel fraction over the low sandy terrace. The position of the boundary between gravel and sand sediments was surveyed along the beach profile. Those measurements allowed us to track the movement of the gravel interface under the action of waves and tides and relate those to the tidal cycle. The position of the interface gravel-sand has a linear relation with the position on the beach elevation and on the cross-shore distance giving a foreshore beach slope within the range 1:6 and 1:8, typical of mixed and gravel beaches (Van Wellen et al., 2000), see Fig3. Thus, at this site it is assumed that the shingle upper beach appears to determine the performance of the beach as a defence and, furthermore, that the sand and shingle appear to behave as two distinct morphological systems, with the shingle overlaying a compacted and stable horizon of sand.

3. Methodology

In a review of the LST equations for gravel beaches, Van Wellen et al. (2000) point out the lack of data and information related to coarse and mixed beaches due to the limitations of deploying delicate instrumentation on energetic and erosive shingle shores. Van Wellen et al. only noted three studies of relevance on shingle beaches in the South UK: Nichols and Webber (1987) and Nichols and Wright (1991) at Hurst Castle Spit (1981 & 1982) and Hengisbury Head respectively; Chadwick (1989) at Shoreham Beach. Nicholls et al. (1987) and Chadwick (1989) have attempted to estimate the LST rate for those coarse grain size beaches, using models derived from the CERC equation. Tracers and traps experimental data have been used to calculate *K*, giving values of 7% and 9%, (Nicholls et al. (1987) and Chadwick (1989) respectively), of those obtained for sandy beaches.

A novel methodology to calculate the longshore transport rate for a mixed beach, Milford-on-Sea, has been developed for the first time in the UK. This method consisted on the application of an impoundment technique where a temporary groyne has been deployed acting as a barrier to the sediments. Bodge (1987) and Bodge and Dean (1987) (Wang et al., 1999) have applied an impoundment technique on sandy beach es and they highlighted its applicability to study the LST in other coastal environments, and to the authors' knowledge this work is the first to apply the technique in the UK and on a mixed beach.

From 28th of September to 25th of November 2007 a temporary groyne was deployed at Hordle Cliff. The experimental structure of 40m length was built up with specially designed Geotextile bags of 1m x1m x 1m and filled with native beach material from the berm (see Fig. 4). Approximately 200 customised geobags, designed to carry 2 Tonnes, were modified with a top closing to keep the sediment contained inside. Subsequent to the experiment the bags were emptied and the contents spread along the beach to return the material to the beach system. The groyne location was chosen to be free from existing structures and overlooked by a low cliff from which an Argus Beach Monitoring System (ABMS) could be used to observe the experiment. USACE, 1992 specifies groyne design in terms of specific wave parameters and tidal range. However in this case, the structure length was dictated by the tidal excursion and position of the sediment interface. The 40m length was selected to ensure that the coarse material from the berm would be totally impounded over the spring tidal range of 2m.

Figure 4.Left: Geotextile bag filled with beach material. Centre: view of structure from top of cliff. Right: Temporary groyne seen from the western side looking eastwards.

Beach profile surveys were conducted daily over a 300m survey grid in extent for the duration of the 2 month deployment of the structure (see Fig. 5). That grid was defined with 15 profile lines on either side of the groyne location spaced at 10m intervals. This distance exceeded that recommended in the SPM (1984) "on the order of two or three groyne lengths" where this length is specified from the beach berm crest to the groyne seaward. A Differential Global Positioning System (DGPS) was used to conduct the surveys at low tide. Contemporary measurements of wave climate were obtained using a Nortek AWAC acoustic Doppler current profiler providing wave height and directional spectrum.

Additionally, tidal observations were taken from RBR TWR-2050 Series gauges located at the groyne head and on a nearby waste water outfall along the coast. This provided tidal elevation and spot measurements of surface elevation and wave period. Additional hydrographic surveys were obtained before and after the experiment to enable accurate wave transformation calculation and assessment of nearshore sediment dynamics. Grain size samples were also taken regularly during the experiment.

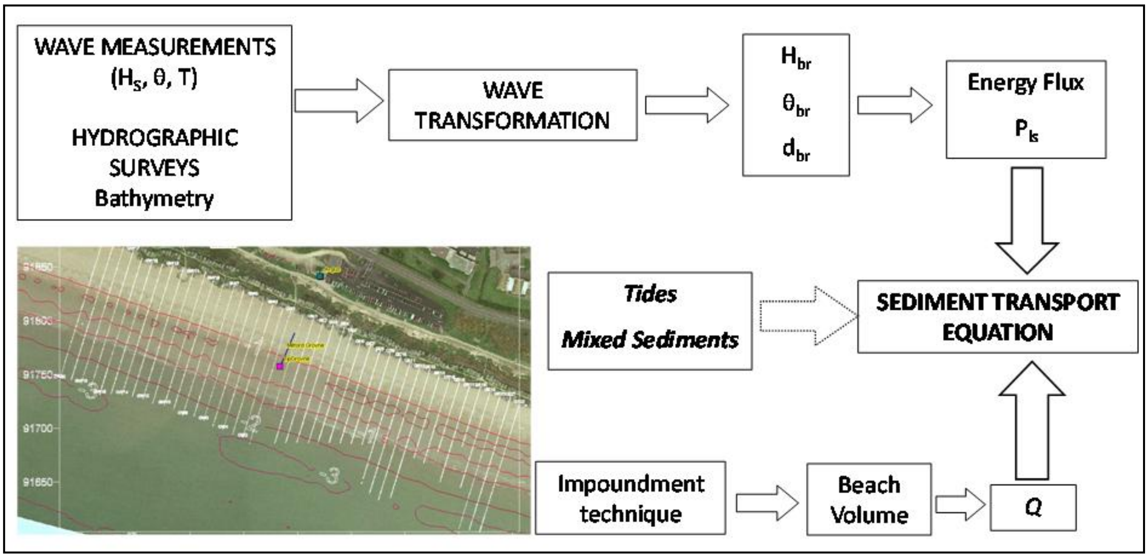

Figure5. Scheme of the project methodology for the calibration. An experiment site image includes overlaid hydrographic data and topographic survey grid.

The impoundment technique then relies on the principle of mass conservation applied with the assumption that the shore normal temporary structure functions as a total barrier for the sediments. Any observed changes in beach volume between two arbitrary profiles lines are then assumed equal to the difference between the sediment flux into and out of the section under consideration. Against the groyne, any volume change is simply equal to the flux towards or away from this barrier. The sediment fluxes in any section must be related to the wave conditions through appropriate LST equations. Then, it is assumed that no sediments pass through the structure and the profile change is due to the longshore transport. Nichols and Wright (1991) noticed that loss of shingle seaward is generally minimal on mixed and gravel beaches and thus this approach can be effective for estimating LST rates.

Further investigations to attempt reliable estimations of the LST rate applying the CERC formula for different locations identified the need of modify the equation considering other parameters that affect sediment transport processes. Those formulae proposed include beach slope and sediment size (Kamphuis et al., 1986), later on wave period or wave steepness (Kamphuis, 1991) (in Kamphuis 2000) or wind, tide and breaking wave (Bayram et al., 2007).

4. Modelling

4.1. One-line model

In the one-line model theory, a line *y(x, t)* is chosen at specific elevation, and the location is evolved with time, using Equation (1). This equation relates the rate of change of location of the line to the sediment transport rate.

$$\frac{dy}{dt} = -\frac{1}{d+B}\left(\frac{\partial Q}{\partial x} + q\right) \tag{1}$$

In this equation *d* is the depth of closure (the offshore depth beyond which longshore sediment transport does not affect the beach), *B* is the berm height (the highest elevation above the still water line which is affected by sediment transport), *Q* is the sediment transport rate and *q* represents sources or sinks along the coastline. Note that x-axis is oriented in the longshore direction and the y-axis in the offshore direction.

If the sediment transport *Q* is known at a discrete number of equally spaced points (0, 1, …, n, n+1) along the coastline section of interest, then the location of the line *y(x, t)* is defined at the mid-intervals of these points (0…n). Fig. 6 shows a schematic diagram for the one-line model, and the numbering system used for the line location and corresponding sediment transport values.

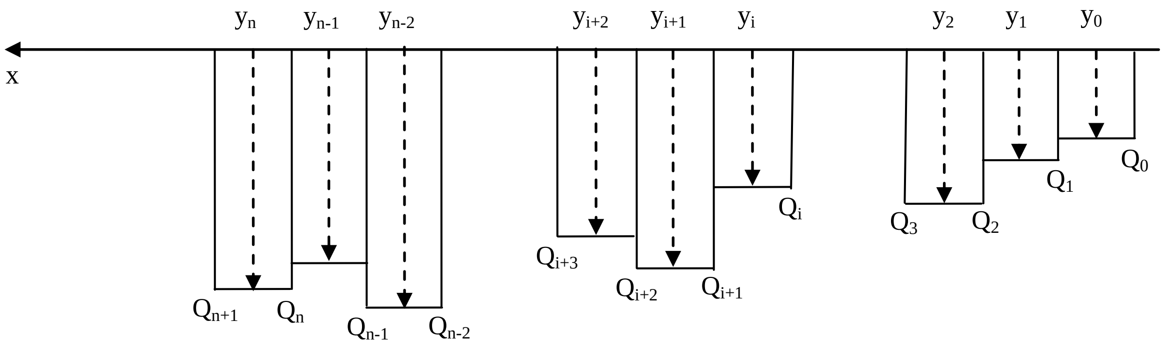

Figure6. Schematic diagram for one-line model.

In order to use equation (1) to evolve the line location, knowledge of the sediment transport rate, *Q*, at each point on the line at any particular time is required. The most commonly used equation for modelling the longshore sediment transport induced by breaking waves is the CERC equation, given by Equation (2):

$$Q = \frac{K}{(\rho_s - \rho)(1-p)g}\left(EC_g\right)_b \sin\theta_b \cos\theta_b \tag{2}$$

Where *K* is a constant to be determined from site specific calibration, ρ_s is the density of the sediment, ρ is the density of the water, *p* is the sediment void ratio, *g* is the gravitational acceleration, *E* is the wave energy, C_g is the wave group velocity, θ is the angle between the incident wave and the shoreline, and the subscript *b* denotes properties at the wave breaking line.

In equation (2) the wave energy can be represented using the Equation (3):

$$E = \frac{1}{8}\rho\, g\, H_b^2 \tag{3}$$

Where H_b is the wave height at breaking and in shallow water on a mildly sloping beach the wave group velocity can be approximated using equation (4) where *h* is the water depth at breaking (Dean and Dalrymple, 2002):

$$C_g = \sqrt{g\,h} \tag{4}$$

A simple forward-difference formula is used to evaluate the $\partial Q / \partial x$ term in equation (1), so that it is evaluated at a point which is centered at the corresponding line location point as:

$$\left.\frac{\partial Q}{\partial x}\right|_i = \frac{Q_{i+1} - Qi}{\Delta x} \tag{5}$$

Higher order differencing schemes could be used, but with the line location and sediment transport rate grid points defined as shown in Fig. 6, they must produce a sediment transport longshore gradient $\partial Q / \partial x$ that is centred at the appropriate line location. Also, when incorporating structures such as groynes, where the sediment transport rate is set to zero, a simple forward difference must be used, otherwise calculated values of $\partial Q / \partial x$ at points adjacent to the groyne point will be influenced by effects on the other side of the structure.

4.2. Wave Model

In order to obtain the information (wave height, wave direction and water depth) at the breaking line needed in the CERC sediment transport formula, equation (2), we run a sophisticated wave transformation model developed by Li (1994). This is a combined refraction/diffraction model that uses offshore experimental data as an input and its solution provides the required breaking line values. The changes in the beach position predicted by the one-line model were used to update the bathymetry on each time step. Also, tidal effects were taken into account by using tidal data to update the water depth at the computational grid points.

4.3. Simulations

Simulations were carried using different lateral boundary conditions at the ends of the region over which the one-line model was applied. The options considered were: (i) fixing the line at its original location, (ii) applying a $\partial y/\partial x = 0$ condition at the end points, (iii) setting a zero transport rate at the ends, (iv) setting $\partial Q/\partial x = 0$ at the end points and (v) letting the line at the ends of the region evolve naturally.

It was found that fixing the line at the end points led to oscillations in the predicted beach profile, initially near the ends but then propagating over the whole region with time. Since a forward difference form must be used to compute $\partial Q/\partial x$ (see equation (3)), then applying a $\partial Q/\partial x = 0$ type condition by setting equal Q values at the ends (e.g. $Q_0 = Q_1$) also resulted (from equation (1)) in the line at the ends remaining fixed and the problems with oscillations mentioned previously were then encountered. After these investigations, the most accurate simulations were achieved by letting the line evolve naturally at the ends of the region. However, the breaking line information obtained from the wave model was modified at the ends of the simulation region, so that values of wave height, wave direction and water depth at the five computational points nearest to the end were set to be equal to the value at the fifth point from the end.

One-line model simulations of the beach profile, at a 1m elevation above the mean water line, have been carried out over a six week period (30/11/07 to 15/11/07) using the CERC sediment transport equation. In these simulations the wave model was run using offshore data at 1 hour intervals. The breaking line information obtained was then used in the one-line model to update the line location every hour. Fig. 7 shows a comparison of the predicted and experimentally observed beach evolution around the groyne over this time period. In these simulations a K value of 0.08 is used in the CERC formula. It is seen that the displacement in the line either side of the groyne is well predicted, with a shift of approximately +/- 2m on either side of it. Note that the scale on the vertical axis in Figure 7 is with respect to the coordinate system used in the coupled one-line/wave model.

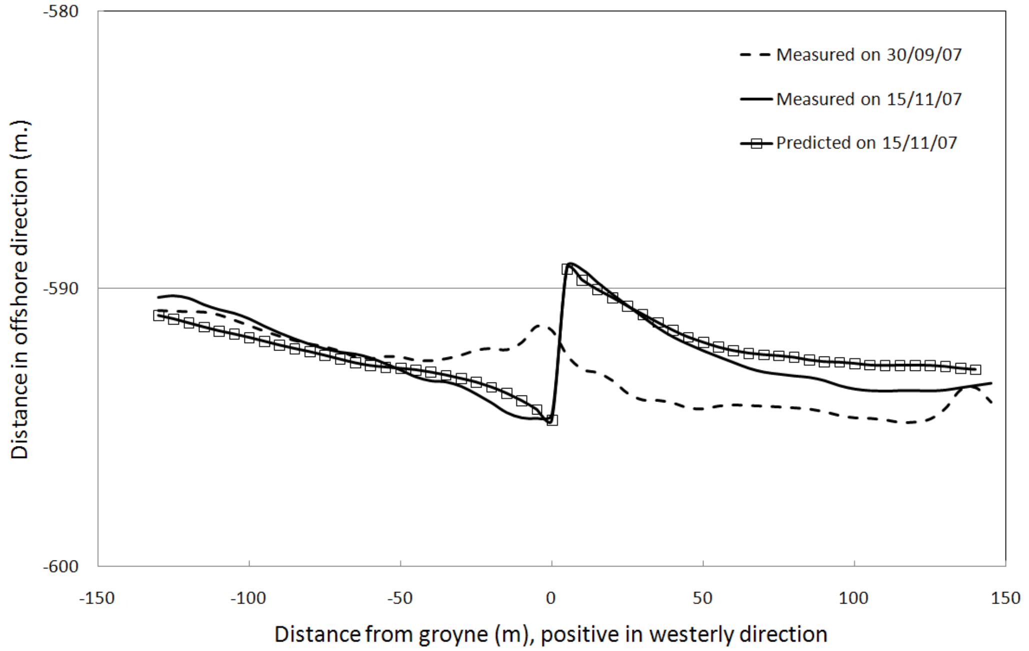

Figure7. Comparison between one line model prediction (using CERC model, k = 0.08) and measured changes in beach profile at a 1m elevation.

In Fig. 8 the sensitivity of the simulations to the CERC equation K value is considered. Here the predicted beach profile is shown for K values of 0.06, 0.08 and 0.10. On comparison with the measured profile on 15/11/2007 shown in Figure 8, all three values of K give reasonable predictions in the displacement of the line at the groyne. It is seen that, as expected, the predicted shift either side of the groyne is reduced when K=0.06 and increased for K=0.10.

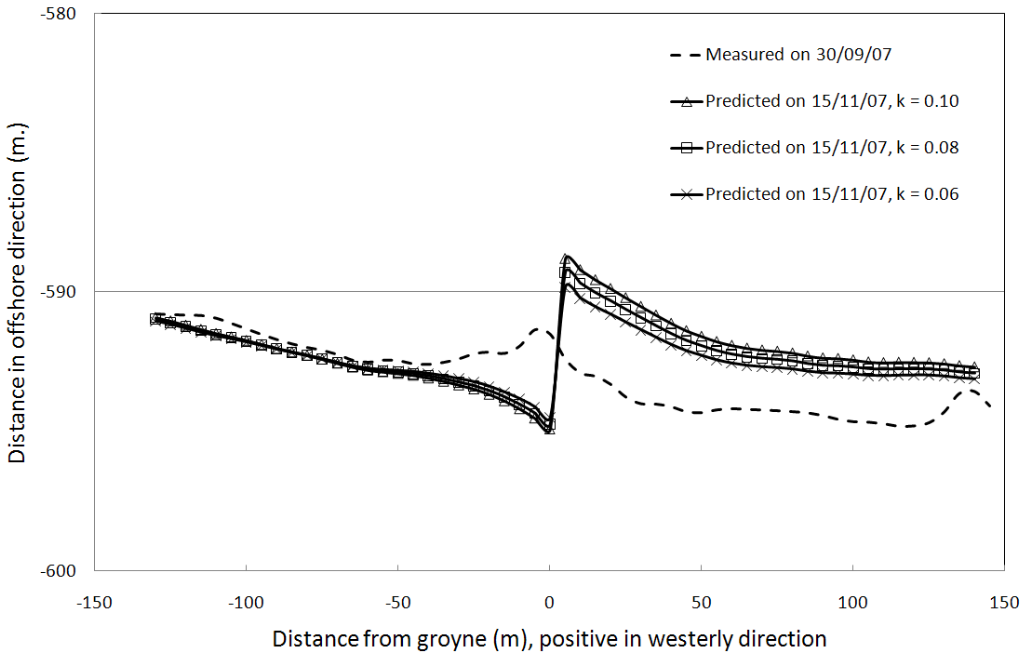

Figure8. Comparison between one line model predictions of the beach profile at a 1m elevation, for various values of the K constant in the CERC sediment transport equation.

5. Preliminary Results

Estimations of LST rates have to rely on data collected by tracers, traps and profile or shoreline changes due to the lack of field data on coarse grain beaches (Van Wellen et al. 2000). Tracers and traps tend to overestimate the LST and present limitations regarding to the shingle sediment size and it is associated with more variability in space and time rather sandy beaches. Some authors as Van Wellen et al. (2000) and Wang et al. (1999) have pointed out the most appropriate and reliable method for assessing the beach response in the long term and also from the engineering point of view seems to be an impoundment technique.

Experimental data measured with this technique are being used to calibrate the constants in the CERC formula for the sediment transport rate. Comparisons with contour maps produced from beach deformations that occur due to the presence of a groyne. The values of K obtained here are in general agreement with the published reduction in K for coarse grain sediments, varying in a range between 10% and 15% of that for sand (Van Wellen et al., 2000). The model, thus calibrated, will then be used to assess future evolution of this shoreline over the remainder of the observation period.

Acknowledgements

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IMPOUNDMENT TECHNIQUE TO CALIBRATE LST RATE FORMULA ON A MIXED BEACH

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Abstract: Estimations of Longshore Sediment Transport rates are particularly important for coastal engineering schemes due to the fact that they constitute a dominant broad scale process for the prediction of long term and large scale changes on the beach (Horikawa, 1988). LST formulations are generally empirical and have been developed for sandy beaches (Van Wellen et al, 2000) with little data for mixed and gravel beaches. Up to date only few investigations (Nicholls et al., 1987; Chadwick, 1989; Morfett, 1989) have attempted estimate LST rate for coarse grain size beaches derived from the CERC equation (SPM, 1984). Tracers and traps on shingle beaches in the South UK to estimate *K* have been giving values of 7% and 9% order of magnitude lower, Nicholls et al. (1987) and Chadwick (1989) respectively, than those obtained on sand beaches. The harsh conditions of gravel coastal environments prohibit deployment of delicate instrumentation. Consequently alternative approaches to data collection are requiered. In this work a groyne-impoundment technique has been applied for the first time on a mixed beach to calibrate the CERC equation using field data. Volumetric estimations from beach profile surveys and wave measurements recorded at the field site, Milford-on-Sea (South UK), are being used to estimate the LST rate.

INTRODUCTION

Mixed (sand and shingle) and gravel beaches are generally rare worldwide. However, along the south coast of the UK, they are particularly common coastal features (Komar, 1976). From an engineering point of view, beaches constitute the best natural buffer to protect coastlines against flooding and coastal erosion (Simm

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et al, 1996). Shoreline Management Plans (SMPs) in the UK are non-statutory documents provided by the government to local coastal authorities that provide guidelines for sustainable use of their littoral environments. Thus, in order to improve those plans the design of new regulations demands a better understanding of the coastal processes and beach morphodynamics in particular, in order to predict shoreline changes over the long term + 70 years. The widely known one-line models have practical capability and have demonstrated to predict shoreline change in long term (Dabees et al., 1998).

Notwithstanding, despite being of considerable interest to coastal authorities, to date, there have been few investigations related to coarse sized sediments (López de San Roman-Blanco, 2003). Gravel and mixed beaches are distinguished from sandy beaches because they generally present a steep gravel face, between 1:6-1:8. Consequently, there is a narrower surf zone where refraction processes take place (Kirk, 1980). This type of beach is usually characterized as being reflective due to the high permeability of the shingle fraction (Van Wellen et al., 2000). The harsh conditions of these coastal environments confine the deployment of delicate instrumentation for data recording and as a result there is a scarcity of measurements for gravel and mixed beaches to predict their morphodynamic behaviour over long time scales (Van Wellen et al., 2000; Bradbury et al., 2003).

The work presented in this paper consists of a novel impoundment technique based on a groyne experiment that has been carried out for the first time in the UK and on a composite beach. Regular beach profile data, wave and tide measurements have been recorded during an intensive field data collection campaign over the period of two months at a mixed beach study site.

FIELD SITE

The site of the study is Milford-on-Sea located in Christchurch Bay in the South of the UK, Figure1. It is a natural mixed (sand and shingle) beach and it is subject to predominant SSW wave direction and semi- diurnal tides with a spring tidal range of 2m O.D. (Ordnance Datum). As it has been determined by a sediment transport study (SCOPAC, 2004), Christchurch Bay has a relatively recent geological origin and its configuration has been formed by the coastline retreat during the mid to late Holocene transgression in the Quaternary Period.

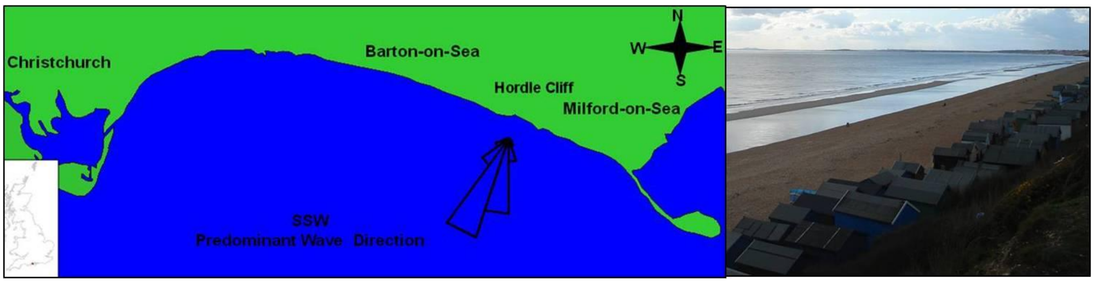

Fig. 1. Left-hand panel: Field Site location, South UK. Right-hand panel, Milford-on-Sea view from Hordle Cliff, bar exposed during spring tide

In one hand, the field site has an important value from the natural conservation point of view because the Coastline from Barton-on-Sea to Hurst Spit has been designated a Site of Special Scientific Interest (SSSI), particularly Milford-on-Sea, is a Coastal Area of Outstanding Natural Beauty (AONB). On the other hand, from a research and engineering management point of view the interest of the area lies on the different coastal elements that are presented: natural beach at Hordle Cliff, longshore sandy bar exposed during spring tides, beach cusps under low energy and normal incidence wave conditions, cliff eroding at the western side near Barton-on-Sea, and coastal defence structures comprising timber groynes and seawall between Milford-on-Sea and Hurst Spit. Thus, in terms of coastal planning, Milford is subject to New Forest District Council and is covered by the Solent Strategic Guidance Plans and Local Authority Coastal Management Plans.

METHODOLOGY

Normally, estimations of LST rates are important for coastal engineering practice because they are intimately related to beach morphodynamic changes over long term time scales under the action of hydrodynamic processes (Simm et al., 1996). Different methods exist for calculate LST rates in the field such as tracers and traps. However, some empirical results obtained applying these techniques tend to overestimate the LST of shingle due to experimental difficulties (Van Wellen et al., 2000). Hence, some authors as Van Wellen at al. (2000) and Wang et al. (1999) have pointed out the most appropriate and reliable method for assessing the beach response in the long term seems to be an impoundment technique.

Groyne Experiment

From 28th of October to 25th November of 2007 an intensive field experiment took place in Hordle Cliff. The experiment consisted of the deployment of a temporary structure constructed from geobags with the aim of acting as a barrier for the sediments on the longshore direction. The geobags of 1mx1mx1m were specially designed to carry 2 Tonnes of native beach material from the berm. These were customized by modifying the top closing to keep the sediment contained inside, Figure 2. Once the experiment had finished, the bags were emptied and the contents spread along the beach to return the material to the beach system.

Fig. 2. Left and right: view of structure from top of cliff during deployment. Centre: geobag filled with beach material.

The Shore Protection Manual (1984) specifies groyne design in terms of specific wave parameters and tidal range. Nevertheless, in this study, the tidal excursion and position of the sediment interface between sand and shingle dictated the structure length being approximately 40m. Previous analysis of beach profile data allowed the tracking of the movement of the shingle fraction along the beach cross-shore section. Thus, the length of the groyne was selected to ensure that the coarse material from the berm would be totally impounded over the spring tidal range of 2m.

A 300m alongshore survey grid was set up in order to conduct the beach profile surveys, Figure 3. On either side of the groyne location 15 profile lines spaced 10m intervals were survey daily during one of the ebb tides. This distance exceeded that recommended in the Shore Protection Manual (1984) “on the order of two or three groyne lengths” where this length is determined from the beach berm crest to the groyne seaward. To conduct the beach surveys a Differential Global Positioning System (DGPS) was used. A Nortek AWAC Acoustic Doppler Current Profiler was used to obtain contemporary measurements of wave climate providing wave height and directional spectrum. Additionally, two RBR TWR-2050 Series gauges were located at the end of the temporary groyne head and on a nearby waste water outfall along the coast. These provided tidal elevation and spot measurements of surface elevation and wave period. To calculate an accurate wave transformation and assessment of nearshore sediment dynamics additional hydrographic surveys were obtained before and after the experiment. Also grain size samples were collected frequently during the field campaign.

Fig. 3. Survey grid layout at the field site. The groyne location is indicated by the blue line. A hydrographic survey before the experiment overlapped

Calibration

In order to apply a LST formula to a specific site, normally, it is necessary to calibrate it against data measured in that particular site and consequently obtain a coefficient value for that beach (Morfett, 1989). The CERC equation (SPM, 1984) is one of the most commonly used formula for modeling the longshore sediment transport induced by breaking waves and is given by Equation 1:

$$Q = \frac{K}{(\rho_S - \rho)(1-p)g} (EC_g)_b sin\theta_b cos\theta_b \quad (1)$$

Where K is the constant to be determined from site specific calibration, ρ_S and ρ is the density of the sediment and the sea water respectively, p is the void ratio, g is the gravitational acceleration, E is the wave energy, C_g is the group velocity and θ is the angle between the incident wave and the shoreline. The subscript b denotes properties at the wave breaking line.

In Equation 1, the wave energy E and the group velocity C_g are represented by Equations 2 and 3 respectively:

$$E = \frac{1}{8}\rho g H_b^2 \quad (2)$$

$$C_g = \sqrt{gh_b} \quad (3)$$

Where H_b is the wave height at breaking and h_b is the water depth at breaking. Then, for calibration purpose, the CERC equation is formulated in terms of the immersed weight transport rate I_l, Equation 4 (SPM, 1984):

$$I_l = KP_l \quad (4)$$

Where K is the dimensionless coefficient to be determined and P_l is the longshore energy flux given by Equation 5:

$$P_l = (EC_g)_b sin\theta_b cos\theta_b \quad (5)$$

However, estimations of different coefficient values for different sites using the CERC equation demand the necessity of field data available to get reliable results based on this approach (Morfett, 1989; Van Wellen et al., 2000). Thus, assuming the shore normal temporary structure functions as a total barrier for the sediments, the impoundment technique is based on the principle of mass conservation and no sediments are passing through the structure. Nichols and Wright (1991) noticed that loss of shingle seaward is generally minimal on mixed and gravel beaches and thus this approach can be effective for estimating LST rates.

Beach profile survey data are used to calculate the LST rates and relate those to the wave energy flux that is obtain using the wave height and wave incident angle measured also at the field site. Wave transformation model enable us to obtain wave parameters at breaking. Preliminary results are giving values of K in general agreement with the published reductions in K for coarse grain sediments, varying in a range between 10% and 15% of that for sand (Van Wellen et al., 2000).

CONCLUSIONS

This work has demonstrated the applicability of a novel impoundment technique for a mixed beach. The technique evidences a good data set collected for optimising K against a one- line model. Generally, the better the quality of the input data, the better the accuracy of predictions produced (Dabees et al., 1998). The next step is to use the experimental data measured with this technique for calibrating the constants in the CERC equation to allow predictions of LST.

Future work is being carried out to develop a conceptual model for this type of mixed beach, develop a new formula for the LST rate and assess the applicability of the formula for mixed beaches.

ACKNOWLEDGEMENTS

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Calibration of a longshore sediment transport formula on a mixed beach

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In the UK gravel and mixed (sand and shingle) beaches are common coastal features along the south coast and these have a great significance for the protection of coastal communities and environmental and agricultural sources (Mason et al., 2001). Up to date only few investigations (Nicholls et al., 1987; Chadwick, 1989) have attempted estimate LST (Longshore Sediment Transport) rate for coarse grain size beaches derived from the CERC equation (SPM, 1984) that is one of the most familiar formulae in coastal engineering which has been developed for sandy beaches. Tracers and traps experiment data measured on shingle beaches in the South UK were used to calculate *K* giving as a result values of 7% and 9% times lower, Nicholls et al. (1987) and Chadwick (1989) respectively, than those obtained on sand beaches.

In this work an impoundment technique has been applied for the first time in a mixed beach to calibrate the CERC equation using field data. Volumetric estimations from beach profile surveys and wave measurements recorded at the field site, Milford-on-Sea (South UK), are being used to estimate the LST rate.

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Determining littoral transport rate on mixed beaches using an impoundment technique. Proceedings of the 31st International Conference in Coastal Engineering Martín-Grandes, I., Simmonds, D.J., Kizhisseri, A., Chadwick, A.J., Reeve, D.E. and Davidson, M.

DETERMINING LITTORAL TRANSPORT RATE ON MIXED BEACHES USING AN IMPOUNDMENT TECHNIQUE

Inés Martín-Grandes[1], David J. Simmonds[2], Abdulla Kizhisseri[3], Andrew J. Chadwick[4], Dominic E. Reeve[5] and Mark Davidson[6]

The determination of the longshore sediment transport (LST) rate plays a fundamental role in any study related to the solution of coastal engineering problems. Detailed knowledge of the LST is necessary for the assessment of the beach evolution in response to the wave climate and due the presence of coastal protection structures. Amongst the most typical formulations that have been proposed to determine the LST rate are the well known CERC formula (SPM 1984), and those proposed by Kamphuis et al. (1986), Kamphuis (1991, 2002) and Bayram et al. (2007). However, most of these empirical equations have been derived from investigations related to sandy environments. In the UK gravel and mixed (gravel and sand) beaches are common coastal features along the South Coast and these have great significance for the protection of coastal communities and environmental and agricultural resources (Mason and Coates, 2001). Yet, only few research efforts have been carried out on this type of beaches (Chadwick 1989). In the present study, an impoundment technique has been employed to measure LST rates in a mixed (gravel-sand) beach with the aim of calibrating existing formulae for estimating the LST developed for sandy beaches.

INTRODUCTION

Coastal environments are natural systems susceptible to change under the actions of coastal processes and the development of communities in littoral boundaries. Indeed, the effect of climate change raises the vulnerability of coasts to damage. Nowadays more information about shoreline changes or beach morphodynamics is requested by local authorities to integrate sustainable activities within the environment under the frame of coastal management plans.

Beaches are the most efficient mechanism for protection the coast and their configuration is the result of the balance between the action of waves and tides and the sediments moving along the coast. In the south of the UK mixed and shingle beaches are common features along the coast, but are less well understood than sandy environments despite being of considerable interest to coastal authorities (López de San Román-Blanco, 2003). Indeed, most empirical equations for longshore transport have been derived from investigations related to sandy environments (Chadwick 1989).

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Quantification of coastal sediment transport, specially the longshore sediment transport, LST, constitutes the most essential information for beach management in coastal engineering. Spatial and temporal changes in LST along a coastline are inextricably linked to beach profile changes over both the short and long term (Horikawa, 1988). Measurements of beach profiles are used to estimate variability in relation to meteorological forcing and to monitor the changes of beach volume and the shoreline position. The latter "line of demarcation between the water and the exposed beach" Komar (1976) is variously defined parametrically (Farris and List 2007).

In the work presented here, a field experiment has been executed to attempt to calibrate the CERC Equation (USACE, 1984) for a mixed beach. Other well known formulations for the sediment transport as Kamphuis (1991) and Van Wellen et al. (2000) will also be looked at.

Mixed beaches are complex systems where the hydrodynamic processes affect different due to the mixture of sediments and their hydraulic properties than pure sand or gravel beach (Kirk, 1980; Mason et al. 1997). Again examples of research into mixed beaches in terms of their dynamics and predictive tools for their behaviour are few and far between: Kirk, 1980; Mason et al., 1997; Mason et al., 2001; Bradbury et al., 2003; López de San Román-Blanco, 2003; López de San Román-Blanco et al., 2006; Buscombe et al., 2006. In these, there is much mention of the need for further research, including the calibration of existing methods derived from laboratory experiments or physical models.

We thus present our study of a mixed (gravel and sand) beach on the south UK coastline. The aim here is to measure the littoral transport and develop a modelling approach for understanding the longer-term shoreline behaviour at this beach.

FIELD SITE

The field site is Milford on Sea, located in Christchurch Bay (Hampshire, UK), see Fig. 1. The work forms part of a project, RF-PeBLE: Risk-based Framework for Predicting Long-term Beach Evolution. This stretch of coastline contains many typical coastal elements: cliff erosion to the western side near Barton on Sea; natural beach at Hordle Cliff; coastal defence structures comprising timber groynes and seawall between Milford on Sea and Hurst Spit. The coastline from Barton on Sea to Hurst Spit is also designated a Site of Special Scientific Interest (SSSI). Milford on Sea is a Coastal Area of Outstanding Natural Beauty (AONB) (SCOPAC, 2003). In terms of coastal planning, Milford is subject to New Forest District Council and is covered by the Solent Strategic Guidance Plans and Local Authority Coastal Management Plans.

A sediment transport study (SCOPAC, 2003) revealed the relative recent geological origin of Christchurch Bay. Its configuration has been formed by the coastline retreat during the mid to late Holocene transgression in the Quaternary Period. Sediment inputs come from cliff or coastal slope erosion of gravel, sand

and clay and in general the littoral drift pattern is west to east transporting gravel and sand in the bay. Also onshore-offshore sand transport is seen in Hordle Cliff.

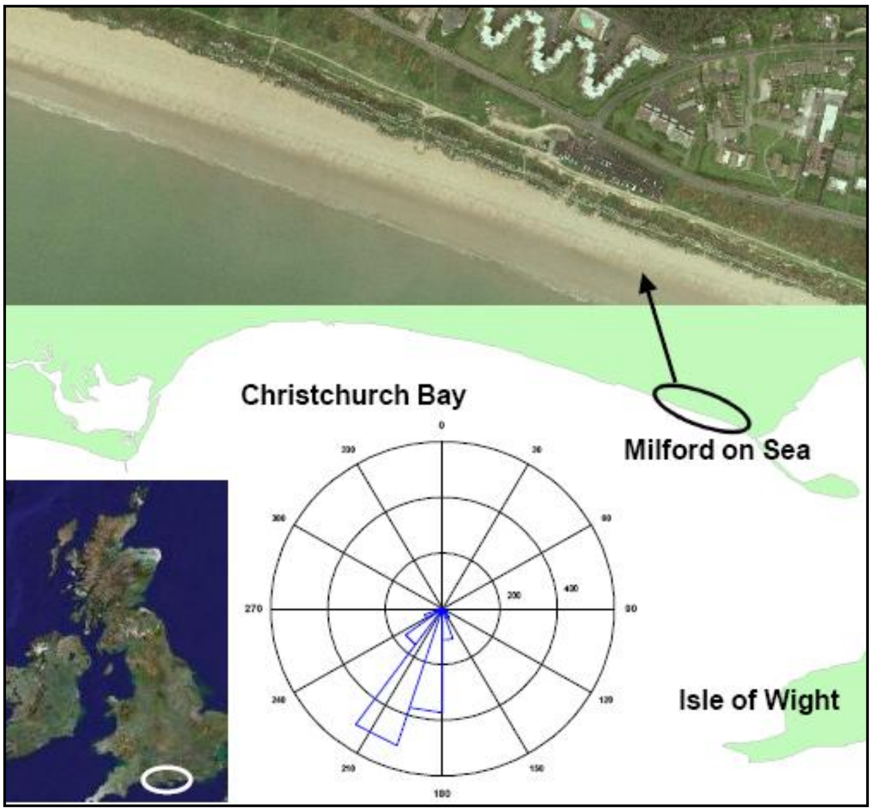

Figure1. Field site location, Christchurch Bay, South U.K. (Google; SANDS Database, Halcrow)

Focusing in on the Hordle Cliff beach , see Fig. 2, this can be seen to be of shingle and sand, dominated by an upper berm of coarse shingle which acts as the main toe protection for the cliffs. Well-defined cusps are often evident on this berm. Below the berm is a sandy terrace with a bar that is exposed at low tide. Two further bars are evident in energetic wave conditions.

The predominant wave direction is SSW and the mean tidal range of 2m is low in comparison with other coastal zones around UK. This and the double tide cycle may be due to the combination of shallow waters and the location near to an amphidromic point (Komar, 1976). During a tide the beach can be considered to move from being reflective at high tide to dissipative at low tide in line with

Davidson et al's (2004) observations. This changes the nature of processes acting upon the berm and those acting on the lower terrace.

Coarse grained beaches are characterized by their narrow, high energy surf and swash zones over a steep, reflective beach face (Kirk, 1980). The reflective nature is related to the size and hydraulic properties of material. The permeability controls the beach slope, which in turn controls the breaker type, which is typically surging or plunging for mixed beaches (Van Wellen et al., 2000; López de San Román-Blanco, 2003). At low tide, dissipative conditions are evident with the wider surf-zone typical of a sandy beach.

Figure2. Milford on Sea: on the top, left-hand panel, sand bar exposed during spring tide; right-hand panel, experimental structure and cusps formed at the beach face; at the bottom, mixed sediments.

METHODOLOGY

In a review of the LST equations for gravel beaches, Van Wellen et al. (2000) point out the lack of data and information related to coarse and mixed beaches due to the limitations of deploying delicate instrumentation on energetic and erosive shingle shores. They noted only three studies of relevance: Nichols and Webber (1987) and Nichols and Wright (1991) at Hurst Castle Spit (1981 & 1982) and Hengisbury Head respectively; Chadwick (1989) at Shoreham Beach.

Because the lack of field data on coarse grain beaches, the estimations of LST rates have to rely on data collected by tracers, traps and profile or shoreline changes (Van Wellen et al. 2000). In general tracers and traps tend to overestimate the LST and present limitations regarding to the shingle sediment size and it is associated with more variability in space and time rather sandy beaches. They conclude that the most appropriate and reliable method for assessing the beach response in the long term and also from the engineering point of view seems to be an impoundment technique. In this, a shore- normal structure acting as a barrier is studied as it traps the longshore sediment transport. It is assumed that no sediments pass through the structure and the profile change is due to the longshore transport. Nichols and Wright (1991) noticed that loss of shingle seaward is generally minimal on mixed and gravel beaches and thus this approach can be effective for estimating LST rates.

This approach was adopted in the work presented here.

One of the most familiar formulae for LST in coastal engineering is the CERC equation (SPM 1984) which has been developed for sandy beaches. . This empirical equation is based on the Energy Flux method which considers the immersed weight of the alongshore moving sediment is proportional to the alongshore wave power per unit length of beach, Eq.1, (Kamphuis et al. 1986).

$$I_l = KP_l \tag{1}$$

P_l is the alongshore component of wave power in the breaking zone and it is defined as Eq. 2:

$$P_l = (C\ )_b (E_b)\cos\alpha_b \sin\alpha_b = \frac{1}{16}\rho g H_b^2 C_b \sin 2\alpha_b \tag{2}$$

Further investigations to attempt reliable estimations of the LST rate applying the CERC formula for different locations identified the need of modify the equation considering other parameters that affect sediment transport processes. Those formulae proposed include beach slope and sediment size (Kamphuis et al., 1986), later on wave period or wave steepness (Kamphuis, 1991) or wind, tide and breaking wave (Bayram et al., 2007).

In order to provide a calibration for sediment transport at the field site a temporary groyne was used to impound sediment. The impoundment technique then relies on the principle of mass conservation applied with the assumption that the shore normal temporary structure functions as a total barrier for the sediments. Any observed changes in beach volume between two arbitrary profiles lines are then assumed equal to the difference between the sediment flux into and out of the section under consideration. Against the groyne, any volume change is simply equal to the flux towards or away from this barrier. The sediment fluxes in any section must be related to the wave conditions through appropriate LST equations, see Fig. 3. An assessment of the accuracy of the technique can be achieved by balancing accretion on the up drift side with erosion on the down drift side (Wang and Kraus, 1999).

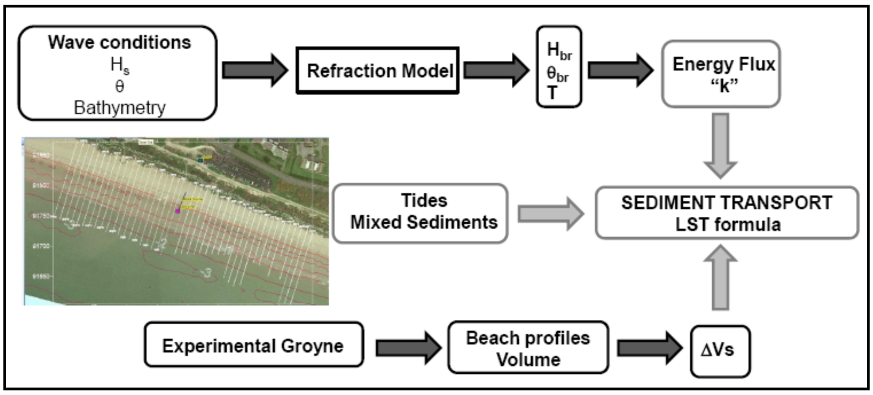

Figure3. Scheme of the project methodology for the calibration including an experiment site image with hydrographic data and topographic survey grid overlaid.

Bodge (1987) and Bodge and Dean (1987) (Wang and Kraus, 1999) have used an impoundment technique to study LST on sandy beaches. However, this work is the first to apply the technique in the UK and on a mixed beach to the authors' knowledge.

The impoundment experiment at Hordle Cliff lasted for 2 months. A location was chosen free from existing structures and overlooked by a low cliff from which an Argus Beach Monitoring System (ABMS) could be used to observe the experiment.

The 40m groyne was constructed directly in front of an Argus camera tower from approximately 200 customised geobags, see Fig. 4. These 1m x 1m x 1m size were designed to carry 2 Tonnes, are of the kind commonly used to transport sediment and granular materials. The design was modified with a top closing to keep the sediment contained inside. The bags were filled up with native beach material from the berm. Subsequent to the experiment the bags were emptied and the contents spread along the beach to fill in the shallow borrow pit from whence it was taken.

USACE, 1992 specifies groyne design in terms of specific wave parameters and tidal range. However in this case, the structure length was dictated by the tidal excursion and position of the sediment interface. The 40m length was selected to ensure that the coarse material from the berm would be totally impounded over the spring tidal range of 2m. It was not practical to trap the sand on the lower terrace. However, at this site it was assumed that the shingle upper beach appears to determine the performance of the beach as a defence and, furthermore, that the sand and shingle appear to behave as two distinct morphological systems, with the shingle overlaying a compacted and stable horizon of sand (see below).

Figure4. Top left: Geobag filled with beach material. Top right: view of structure from top of cliff. Bottom left: western elevation. Bottom right: view shorewards showing accretion-erosion across the structure with some damage evident.

A survey grid, 300m in extent, was defined with 15 profile lines on either side of the groyne location, spaced at 10m intervals. This distance exceeded that recommended in the SPM (1984) “on the order of two or three groyne length where this length is specified from the beach berm crest to the groyne seaward. Surveys were conducted daily over this grid for the duration of the 2 month deployment of the structure, 28th September to 24thNovember 2007. Surveys were conducted at low tide using a Differential Global Positioning System (DGPS). Contemporary measurements of wave climate were obtained using a Nortek AWAC acoustic Doppler current profiler providing wave height and directional spectrum.

Additionally, tidal observations were taken from RBR TWR-2050 Series gauges located at the groyne head and on a nearby waste water outfall along the coast. This provided tidal elevation and spot measurements of surface elevation and wave period. Additional hydrographic surveys were obtained before and after the experiment to enable accurate wave transformation calculation and assessment of nearshore sediment dynamics. Grain size samples were also taken regularly during the experiment.

The groyne was pulled out of the beach on 26th November 2007, the sediment replaced and the beach regraded.

PRELIMINARY RESULTS

The slope and shape of the beach face is a function of grain size, sorting, wave energy and tidal range (Komar, 1976). As mentioned above, we concentrate on the dynamics of the gravel upper beach, which appears to determine the function of the beach as a coastal defence. Thus the LST we are interested in is that of the gravel and the sandy bar and lower beach is not considered. That is not to say that the bar does not have noticeable influence on the shoreline position (Farris and List, 2007).

Having decided this, the changing volumes of sediment need to be determined either side of the groyne in order to infer the LST. We adopted the approach of defining a master profile above which the cross-sectional area at each profile line is calculated. The base of the master profile was found to be well described by a compacted sand horizon observed during excavation of the beach during groyne installation. Furthermore this can be demonstrated by consideration of the position of the seaward limit of the gravel for each profile line over time, ie where the sand "outcrops" the gravel. A good linear correlation (R^2=0.97) is observed in this position indicating a sand beach/horizon slope of about 1:7.

Fig. 5 represents the area changes at the MHWS (Mean high water spring) of the beach profiles respect to the mean area for the survey grid for 56 days. Changes are related with the wave conditions in terms of wave angle and alongshore wave energy calculated with the significant wave height at 6m depth. In general on the west side the accretion was more important than on the eastern side, however longshore sediment transport events in both directions were frequent as well as cross-shore events caused by normally incident waves.

With reference to Fig. 5, changes on the beach profiles due to longshore sediment transport can be seen at the start of November up to 17th November. The combination of energetic wave conditions and an angle from the SSW accumulated sediments on the west side and caused erosion on the east side of the groyne. Then, on 18th November, normal wave incidence and storm wave conditions moved the sediment seaward at the same order of magnitude for both sides of the structure.

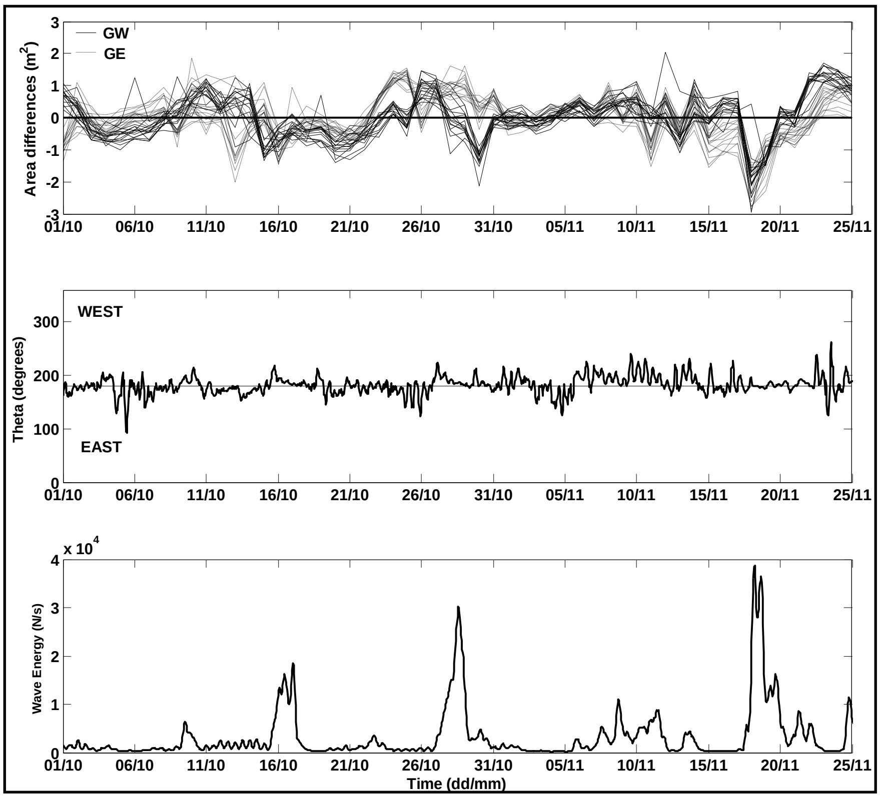

Figure5. Top panel: solid lines represent area changes on the west side of the groyne and dashed lines the eastern profile lines. Middle panel: wave angle corrected respect to the beach orientation, greater than 180° is from the west and less than 180° from the east. Bottom panel: longshore wave energy.

Table 1. Area differences (m^2) respect to the mean and MHWS

Date	GW01	GE01
2102007	0.3885	1.0921
8112007	0.8332	-0.1551
13112007	0.8289	-1.0876
17112007	0.8343	-1.0937
18112007	-2.9437	-2.7836

A significant difference between volume estimations on the updrift and downdrift sides of the temporary groyne for specific wave conditions demonstrates the applicability of the technique, see Table 1 and Fig. 6; in this case the shoreline change is explained by the longshore sediment transport although cross-shore transport is taking place daily with important changes too (Horikawa, 1988).

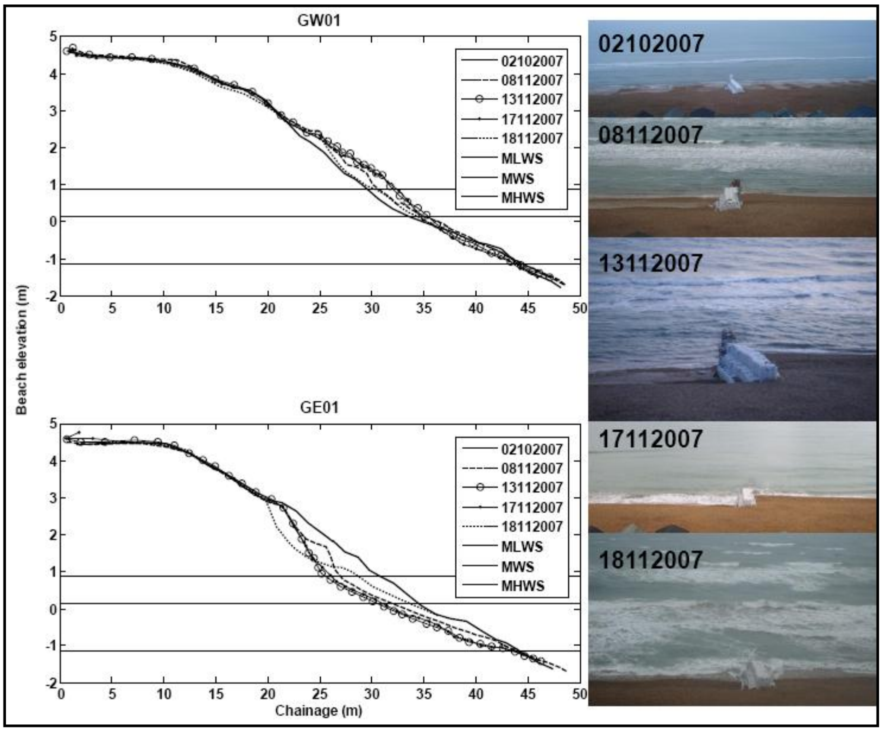

Figure6. On the left-hand panel, graphs represent the cross-section of the beach profile for five dates for the profile lines GW01 (on the west side of the groyne) and GE01 (on the east side of the groyne), both at a distance of 10m alongshore from the experimental structure. Changes shown in the graphs can be compared at the field site with the photos of the right-hand panel.

FUTURE WORK

Future research is focus on the field data analysis in order to develop a methodology for calibration of the formula for the longshore sediment transport rate. From the calibration, a new formula for the LST rate will be derived and fit in a One-line model to predict long-term mixed beach behavior.

As a first approximation, volume calculations are being analyzed in order to calibrate the CERC equation. Afterwards, the aim is to examine the calibration of other formulae, such as Kamphuis (1991, 2002), Bayram et al (2007) and Van Wellen et al. (2000). The latter was developed specifically for gravel beaches and for use in the BORESED LST model (Chadwick 1991, Van Wellen et al, 2000).

Preliminary observations from the beach profile data related to the wave conditions indicate that this technique has worked successfully. This should provide an effective demonstration for the coastal authorities as a good method for evaluating morphological changes and quantifying LST rates. This builds upon Wang and Krauss' (1999) short term impoundment study on a sandy beach. Validation of the new formula with data sets from other mixed beaches will assess its applicability as suggested by Schoonees (2000) for extending its applicability.

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Longshore Sediment Transport
Mixed beach
CERC equation
Calibration
Impoundment technique

Impoundment technique to measure LST Rate on a mixed beach

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Introduction

Mixed beaches (sand and shingle) are generally rare worldwide but are important around the shores of the UK (Mason et al, 1997, 2001) and they constitute the best natural defence to protect littoral environments against flooding and coastal erosion. For the purpose of carrying out a reliable assessment of the shoreline trends it is necessary study the sediment transport patterns and the coastal processes that establish the beach morphodynamic. Estimations of longshore sediment transport (LST) rates are particularly important for coastal engineering schemes and Shoreline Management Plans (SMPs) requested by local coastal authorities. LST formulations are generally empirical and have been developed for sandy beaches (Van Wellen et al, 2000) with little data for mixed and gravel beaches. In this study, a novel groyne-impoundment technique for measuring LST is presented. This was applied to a mixed beach where wave and tide data have also been collected between September to November 2007. Field data will be used to calibrate existing LST formulae, as the well known CERC equation (SPM, 1984) and develop a conceptual model to predict the shoreline evolution of a mixed beach in the longer term.

Methods

To measure longshore sediment transport an impoundment experiment was designed comprising the collection of beach profiles, wave and tide data for two months. The technique consisted of the deployment of a groyne made up of geotextile bags 1m x 1m x 1m filled with native material at a beach located below Hordle Cliff, Milford on Sea, South UK. Based on the principle of mass conservation and given an alongshore direction it was assumed that the longshore sediment transport rate is given by the accretion on the updrift side of the experimental structure and there is erosion on the downdrift side. In order to measure the topographic surveys a grid of 300m length had to be set divided in 15 profile lines in both sides of the groyne. A tidal gauge was deployed at the end of the structure and a Nortek ADCP was deployed in 8m depth to seaward.
The field data collected is being used to calibrate existing longshore sediment formula. Furthermore, the existence of mixed sediments and their distribution along the profile and the tides effect will be examined.

Results

Significant short term changes in the profiles and volumes of sediment adjacent to the groyne were observed according to weather changes. Figure 1 shows the anticipated accretion updrift (to west) and cut-back down-drift which will be used to calibrate LST formula and as model test cases.

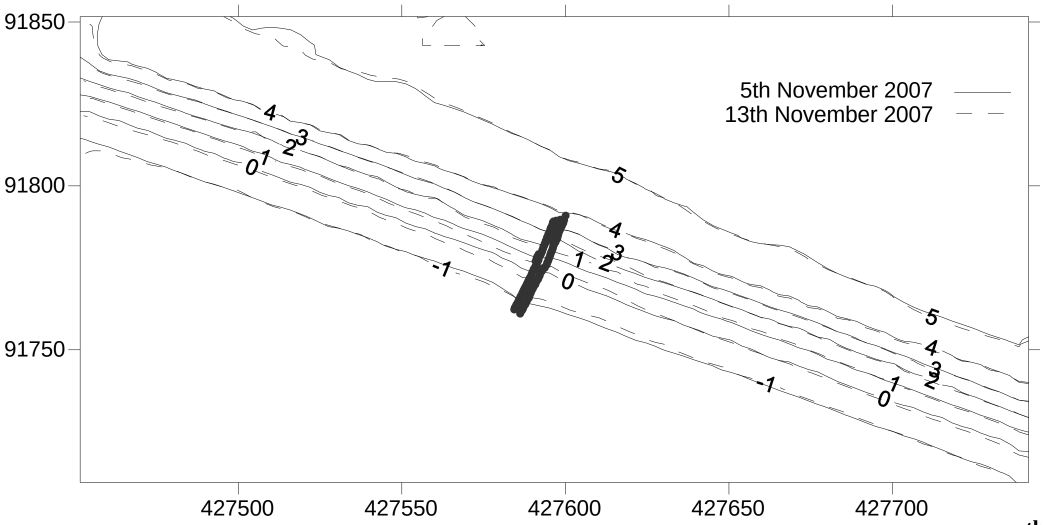

Figure 1 – Contour map and groyne map post for the measurements of 5th and 13th November 2007.

The paper will present the details of this novel experiment and present analytical techniques and preliminary calibrations of the LST using the observed volume changes. The methodology for incorporating such observations into longer-term shoreline management planning will also be discussed.

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Determining littoral transport in a mixed beach using field data

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Predicting coastal evolution typically requires reliable calculations of the LST. Amongst the most typical formulations that have been proposed to determine the LST rate are the well known CERC formula (SPM 1984), and those proposed by Kamphuis (1991, 2002). However, most of these empirical equations have been derived from investigations related to sandy environments. However, in the UK gravel and mixed (gravel and sand) beaches are important coastal features in many locations along the South Coast and these have great significance for the protection of coastal communities and environmental resources (Mason and Coates, 2001). Yet, only few research efforts have been carried out on this type of beaches (Chadwick 1989). The need of further research on coarse-grained beaches has already being identified in a recent review by Van Wellen et al. (2000).

In the present study, an impoundment technique is employed in order to measure LST rates in a mixed beach. A high-quality data set on hydrodynamics and sediment transport as beach surveys with DGPS and sediment sampling are being collected during the field campaign to evaluate the predictive capabilities of the formulae. Field data will be used to calibrate existing LST equation with the aim to develop a model to predict shoreline evolution for mixed beaches.